CONSTRUCTED FACILITIES CENTER

PB98-140767

AN EVALUATION OF THE SKELETAL STRUCUTURE OF THE WEST VIRGINIA INDEPENDENCE HALL, 1859

Contract No. T699-CIDDMOC Subtask E-1

CFC95-225

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Division of Highway
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December 1996

College of Engineering and Mineral Resources

West Virginia University

Technical Report Documentation Page

1. Report No. CFC95-225	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle An Evaluation of the Skeletal Structure of the West Virginia Independence Hall, 1859 Subtask E-1		5. Report Date
		6. Performing Organization Code
7. Author(s) Emory Kemp, Patricia C. Tice		8. Performing Organization Report No. CFC95-225
9. Performing Organization Name and Address Constructed Facilities Center West Virginia University CEMR P.O. Box 6103 Morgantown WV 26506-6103		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. WVDOH T699-CIDDMOC
12. Sponsoring Agency Name and Address West Virginia Department of Transportation Division of Highways 1900 Kanawha Blvd., East, Building Five Charleston, WV 25305-0430		13. Type of Report and Period Covered Final Report 11/9/88 to 9/30/95
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

This project is an evaluation of the structural system of West Virginia Independence Hall, an 1859 Custom House built by the Department of the Treasury in Wheeling, West Virginia. The history of the design organizations and construction is detailed to describe the innovations in materials and design. The floor system is composed of some of the first rolled wrought iron I-beams manufactured in the US. To make the system fireproof, the floor itself is supported by low brick arches that span between the iron beams. Although the system is, in actuality, merely fire resistant, the arches do contribute to the strength and stiffness of the system. Both destructive and non-destructive testing is used to evaluate the material properties of the constituents and an approximation of a level II reliability analysis was formulated to set the limits of the system's capacity. A theoretical model for the behavior of the system is then proposed, detailing the composite nature of the brick and the iron, and the stresses imposed by the construction methodology. The analysis includes a cracked transformed section analysis as well as a three dimensional finite element analysis. The analysis shows that the system has at least 33 percent more capacity than assuming the iron beams support the entire dead and live loads.

17. Key Words		18. Distribution Statement	
19. Security Classif. (of this report)	20. Security Classif. (of this page) Unclassified	21. No. of Pages 112	22. Price

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Chapter 1: Introduction

The analysis of historic structures has received increasing attention by historians, engineers, and preservation professionals, especially in public works and transportation. Legislation over the last twenty years has placed historic buildings and bridges squarely in the realm of cultural resources. Regulations require that any federally funded public work to include the impact on the structure in the Environmental Impact Statement (EIS) and provide for mitigation of any damage to that cultural resource. Therefore it becomes important to have techniques to analyze historic structures in ways that provide the least amount of damage to the structure.

The analysis of this structure is also important in its own right because of the information it yields on design and construction in that time period. Structural details of the Wheeling Custom House provide us information on the manufacturing technology, business environment and the practice of both architecture and engineering in the 1850's.

Little has been done to explore the relative safety of aging materials, some of which may have had questionable properties even at the time of construction. Although past performance can be an indicator of current strength, it cannot be taken as the final word on a structure's continued performance, especially if future performance levels must exceed past structural requirements. Many public works departments currently view the analysis of historic structures as a nuisance, but detailed analysis early in the life cycle of a project may actually save money in two ways. First, the cultural resources portion of the EIS has been the fourth largest cause of delays in getting a project ready for construction. Although the historic importance of a structure does take time to determine, many states have begun to establish priority lists for bridges and buildings that speed up that portion of the process. This leaves the actual structural analysis of the existing structure as the bulk of the remaining time needed to handle the mitigation. Second, many localities have discovered that with detailed analysis and creative construction management, rehabilitation of a historic structure may be much less expensive than replacement. There have been several cases where the cost of rehabilitation has been less than the 20% match required to replace the structure under the federal bridge replacement program. Currently, some of the biggest barriers to reaping the benefits of rehabilitation lie in structural testing technologies and analysis complexity. Any headway made in these two areas will undoubtedly provide substantial long-term benefits.

The focus of this report is on the structural analysis of the West Virginia Independence Hall (1859), which was built as a custom house in Wheeling, West Virginia. The building is important historically because the vote for West Virginia's statehood was held in the third floor courtroom during the Civil War. The current renovations target restoring the building to the grandeur it possessed in 1863 when this historic vote was taken. Additionally, the building functioned as the offices for the governor for a few years. From an engineering point of view, the floor system utilizes iron I-beams manufactured by The Trenton Iron Co.--some of the first to be produced in this cross section. The freestanding nature of the interior skeletal frame is a forerunner for the Chicago School's skyscraper; all it lacks is the curtain wall. In the history of the design of the Custom House we find early examples of mass production and duplication--a pivotal concept for extensive construction at far flung locales. This building's history provides a snapshot of design and construction in the 1850's, an industry comfortable with design based on experimental information that was moving to design based on analytical reasoning.

Object.

The West Virginia Independence Hall is currently being used as a museum of state history, and would be perfectly suited for social functions if it were not for its current low structural rating. To be able to increase the structure's current rating, several goals must be attained:

1. From a historical standpoint the structure's design, construction, and use must be explored to isolate the original designed capacity, any flaws or strengths inherent in the construction, and any previous cases of overloading.

2. As there are no building codes governing the materials involved in this construction, limits must be set in light of the statistical properties inherent in the materials that assure safe use of the structure. Reliability theory will be used to establish reasonable design allowable stresses and loading.

3. The materials involved must be analyzed to determine the statistical properties associated with their material strength and stiffness. This will include both destructive and non-destructive evaluation of the significant constituent materials.

4. The behavior of the system must be examined and appropriate structural models created to determine the system's safe capacity.

Once a safe structural capacity is determined, this must be translated into recommendations of safe usage that can be easily understood by those who use the building.

Scope.

This first chapter deals primarily with the goals and scope of the work. The second chapter focuses on the US Department of the Treasury's building program. After a brief history of the Treasury Department's Office of Construction, the narrative focuses on the personnel in the Treasury at the time, specifically James Guthrie, Captain Alexander H. Bowman and Ammi B. Young. It explores their contribution to this new material, as well as their contributions to the design process and this particular building. The history of Cooper & Hewitt is then explored in Chapter 3, especially where it concerns the rolling of a 9" I-beam, the tests performed on the beams, and the probable theoretical sources of information that could have been available to them to estimate member strength and stiffness. From the information gained in this chapter, a probable load rating as was intended by the designer is discussed. The fourth chapter is a broad history of the building from construction to its current use mainly focusing on the earlier renovations. Some of the later additions are discussed as alterations to the structure and possible overloading conditions. The chapter details the building's history in the context of the current fire proofing and ventilation technologies. This should shed some light on the purpose of using such a unique structural configuration.

To perform an analysis of this floor system, it is important to understand the material properties of the structural components, and this is the focus of the fifth chapter. First, the brick in the jack arch is discussed and the results of the destructive testing are analyzed. The properties of the iron I-beams are then explored both through destructive and non-destructive testing. This testing includes standard structural tensile coupon tests to determine both stiffness and ductility, as well as micrographs of the grain structure and determination of chemical composition. The non-destructive testing includes ultrasonic measurements to determine material stiffness using both standard pressure waves (P-waves) and surface (Raleigh) waves. Conclusions are then drawn about the material properties to be applied to the structural analysis of the floor system.

Chapter 6 outlines the determination of reasonable safe capacity using reliability theory. Determination of reasonable reliability index (β) values is discussed and these are used to establish reasonable resistance factors for the failure modes in question.

The seventh chapter presents a detailed description of the existing structural system, including diagrams based on testing done to determine the as-built condition of the structure. The load path and structural behavior of the system is described as assumed in the structural analysis to follow. The floor system is then isolated as the major determining factor in the capacity analysis, and a cursory analysis of the other major components is detailed to establish their capacities.

The eighth chapter details the structural analysis of the jack arch floor system. Finite element analysis is used to detail the system behavior at factored loads. This model is then used to validate the assumption of composite action between the two components. The second phase of the analysis involves a cracked transformed section analysis of the composite system to determine its ultimate capacity. Stress/strain data are used for both iron and brick to establish the stress state in each component. The ultimate capacity is then evaluated using factored loads and resistance to provide for the level of reliability determined in chapter six.

In chapter nine reasonable usage is discussed in light of the system's capacity.

The concluding chapter provides an overview of the pertinent information in the proceeding chapters and discusses further analysis or structural renovations that might be desirable.

The structural analysis of this building will only address the floor loadings. The other aspects of the structural capacity of the building have previously been examined except for the seismic behavior of the building.

Chapter 2: THE TREASURY DEPARTMENT'S BUILDING PROGRAM

Construction Leadership in a Period of Growth

To understand the environment surrounding the design and construction of the Wheeling Custom House, it is important to look at the Treasury Department's Office of Building and Construction. This agency was responsible for nearly eighty buildings between 1852 and 1862, since there was no General Services Agency for the federal government[1]. Congressional appropriations were made for custom houses, federal court houses, post offices and marine hospitals, all to be constructed by the Treasury Department. The addition of new territory to the US in the first half of the 19th century produced a huge growth in commerce, taking advantage of not only new resources but new transportation routes to get those resources to market. The need for buildings to house federal administrative services for all these new ports and commerce centers required that the Treasury Department reorganize its administrative structure to handle the increased construction volume. In most localities, the building constructed by the Treasury Department served several departments and often comprised the local federal building. The design of dozens of buildings scattered around the country required a balance between the flexibility to meet local needs and the standardization necessary to cope with the enormous workload. In the Treasury Department two major figures were pivotal in balancing these needs: the Supervising Architect, Ammi Burnham Young and the Engineer-in-Charge, Capt. Alexander Hamilton Bowman.

During this time period, the lines between architect and engineer were quite blurred, so it is

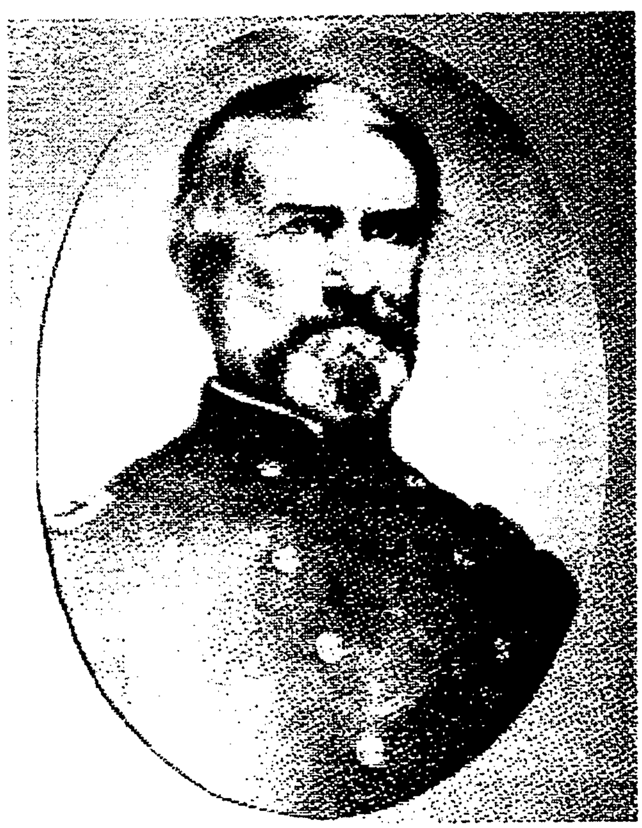
[1]Overby, Osmund R. "Ammi B. Young: an architectural sketch" *Antiques* 81:530-533

somewhat deceiving to place individual labels on them. They were given equal pay, but their duties were quite different in this office. Bowman was largely in charge of site selection and construction supervision, while Young was responsible for the designs themselves.

Although much of the architectural design was performed by Young in the Treasury Department, it is important to note that the manufacturer also played an important role. The team of Bowman and Young was quick to take advantage of new technology, but as with any innovation, the manufacturer is more likely to be familiar with the state of the art in both theory and practice than the designer. Understanding both the Treasury Department and Trenton Iron personnel is important to understanding the design process. The next chapter will go into detail about the manufacturer's role.

Captain Alexander Hamilton Bowman

West Point training. Alexander Hamilton Bowman graduated from West Point in 1825[2]. West Point was established in 1802, to provide officer training for the US Army. Its technical program quickly became a major source of American engineering expertise not only in military engineering, but civil works as



Captain Alexander Hamilton Bowman

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[2]*Register of Graduates and former Cadets of the United States Military Academy*. Assoc of Graduates, USMA. West Point, NY, 1990. p 251.

well. The technical coursework was modeled after the highly acclaimed *Ecole du Ponts et Chausses* in France, and many of the faculty were trained in that tradition or imported directly from France. The Ecole was well known for its theoretical work in all facets of engineering science[3]. In contrast, most engineering in Great Britain and the United States was carried out on a more empirical level. In fact, Thomas Telford and many other engineers of his day preferred an apprentice that worked up through the ranks over a university educated graduates, in large measure because of the questionable state of classical, analytical theory at the time[4]. The *Ecole* was established under the philosophy that students should first be taught the state of the art in science and math so that their design methodologies could flow from that theoretical information. Although the academic work at the *Ecole* was largely theoretical, students were required to spend their summers working on construction sites to gain practical experience. This created a good balance between theoretical knowledge and practical expertise.

Until Mahan's text came into use, West Point used the French text, *Programmes ou Resumes des Lecons d'un Cours de Construction*[5], written by M. J. Sganzin, the Inspector General of Bridges, Roads and Maritime Travel, and a Professor at the *l'Ecole Royale Polytechnique,* a preparatory college in the sciences. The text covers a wide range of subjects from strength of materials to waterworks design. The bulk of the text is concerned with material properties and construction practice. There are a few sections that refer to member testing and the theory of strength of

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[3]Calhoun, Daniel H. *The American Civil Engineer.* The Technology Press, Cambridge MA, 1960. p 40.

[4]Singer, Charles; Holmyard, E.J.; Hall, A.R.; Williams, Trevor. *A History of Technology. Volume 4, The Industrial Revolution, 1750-1850.* The Clarendon Press, Oxford. 1958.

[5]M. J. Sganzin, Troisieme edition, 1821.

materials. These sections summarize the experimental evidence for different design principles and detail a small amount of theory on material behavior. Differential equations for deflections are not discussed in any detail, and ultimate load is taken as the material resistance. No effort is made to distinguish elastic from plastic behavior, therefore, the failure mode for a fixed beam is rightly shown as a three hinge mechanism, although the progression of the hinge formation is not detailed. The text details a triangular bending stress distribution about a neutral axis, but fails to designate where that neutral axis should be[6]. Despite these theoretical failings, *Programmes* covers construction practice quite adequately.

After graduation, Bowman taught geography, history and ethics at West Point for a year, after which he entered active military service[7]. The Corp of Engineers were directly responsible for the waterways of the country and as part of that group, Bowman supervised the construction of several river and harbor improvements. The US Army occaisionally allowed it's officers to be seconded to a civilian organization for the benefit of the country and Bowman was sent to the Treasury Department in 1853.

Bowman's role in the Treasury Department. In 1853 with the appointment of James Guthrie to the post of Secretary of the Treasury, some major changes occurred in the practice of construction for the federal Government. A decade earlier, Young had taken the place of Robert Mills as architect for the Treasury Department. Young had a longstanding relationship with the Treasury, having won several architectural competitions including the U.S. Bank of Burlington and the Boston Custom

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[6]ibid. pp.72-74.

[7]Marshall, Paul and Marshall, Frances J. *Historic Structure Report*, West Virginia Independence Hall, Wheeling Custom House. 1986. p 6.

House, a million dollar project. The existing organizational system was based on these competitions between private architects. The architect chosen would then superintend the construction of that building. During the decade before his appointment as Supervising Architect, Young was responsible for supervising the architects for each of the buildings under construction. This was quickly becoming more than Young could individually administer. In the reorganization, a new office, the Office of Building Construction was created and Young was given the title Supervising Architect. Guthrie then requested that Congress appoint a "scientific and practical engineer" to the post of Engineer in Charge for the building office. In response, The Secretary of War, Jefferson Davis, released Capt. Bowman from the Corp of Engineers to work for the Department of the Treasury[8]. Although Bowman carried the title 'Engineer in Charge', he was equal in rank and pay with Ammi B. Young, the Supervising Architect. As the Engineer in Charge, he handled the selection of sites and the overall supervision of both design and construction for the Treasury Department. All design work was standardized, which was relatively simple since there was only one architect designing for the whole of the Treasury, instead of different individual architects for each project. As a "scientific and practical engineer" he was in a position to encourage engineering innovation, and did so freely, particularly in the case of wrought iron. With the design aspects left in Young's capable hands, Bowman spent the majority of his time "on the road" selecting sites and checking construction. To aid him in this, Bowman required all construction superintendents to have construction photographs taken. Bowman also instituted the practice of publishing the plans of the buildings constructed by the Treasury Department as a teaching tool for West Point and other major educational institutions[9].

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[8]ibid. p 5.

[9]ibid. p 6.

His engineering expertise seemed largely associated with geotechnical concerns. While at the Treasury, there is record of a serious settlement problem in the Marine Hospital at New Orleans that required his personal attention to determine appropriate mitigation measures[10]. Although he was not as closely connected with building design (which was at the time largely an architectural field) he was a staunch supporter of the Cooper & Hewitt system, and initiated the request for member testing to be done to determine both capacity and stiffness. Supervision for these tests was arranged by Bowman using another Corp of Engineers officer, Lieut. B. S. Alexander[11]. It seems he went out of his way to assure that the information needed for the calculations was available to the architect, even if he was not doing the calculations.

Bowman graduated only a year after Dennis Hart Mahan, who eventually wrote the major text used at West Point from his experience studying in France early in his military career[12]. It is likely that he was familiar with both Mahan and his construction text, and it is possible that he passed the information on to Young, although there may have been little available time for the staff at the Treasury Department to put it to use. At one point during Bowman's tenure, he listed 46 custom houses in the course of construction or for which appropriations had been made.[13]

At the time the boundary between civil engineering and architecture was vague at best. In the

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[10]From Bowman, 1855. "Letters sent, chiefly by the SA and the EIC" Sept 1852-Aug 1862. Records of the Public Buildings Service, Record Group 121. The National Archives, Washington, DC.

[11]Alexander, B.S. "Wrought Iron Beams" Ex. Doc. No 56., House of Representatives, 33rd Congress, 2nd Session.

[12]Calhoun, Daniel H. p 40.

[13]Moran, Geoffry P. "The Post Office and Custom House at Portsmouth, New Hampshire and Its Architect, Ammi Burnham Young." *Old Time New England* Vol. LVII, No. 4., p. 89.

Treasury Department it appears that Bowman was largely responsible for executing the design, while Young was responsible for the design itself. Therefore, it was Bowman that supervised the tests on the iron, while Young discussed the properties in more theoretical terms. If any structural calculations were performed at the Treasury, Ammi B. Young, not Capt. Bowman, would have been responsible for them.

Ammi Burnham Young

Early career: A.B. Young has been described by most historians of architecture as a competent practitioner, adept in all classical architectural forms[14]. In the majority of his federal projects Young modeled the designs after the Italian pallazo (palace). This communicates a sense of approachable solidity, the image the federal government wished to portray. His buildings are respectable and expertly executed, although not particularly innovative. He is rumored to have designed his first building when he was nineteen as volunteer work for his church[15]. At 14, he joined his father, a carpenter and surveyor, and it is likely that his architectural and engineering background stemmed from work with the elder Young and collaborations with other current architects like Alexander Parris and Robert Mills[16]. As such, his training was obviously very practically oriented, and any theoretical knowledge he obtained would have been from the journals and carpenter's manuals of the day[17]. His first documentable projects date to 1828 at Dartmouth[18]. By 1830, he was

[14]Wodehouse, Lawrence, "Ammi Burnham Young, 1898-1874" *Journal of the Society of Architectural Historians* Vol.25. p 280.

[15]Overby, Osmund. "Ammi B. Young: an architectural sketch." p 530.

[16]Wodehouse, Lawrence. p 268.

[17]Some possibilities include Tredgold's *Elementary Principles of Carpentry* and Benjamin Hale's *Mechanical Principles of Carpentry* as well as numerous copybooks of the period.

advertising himself as an "architect and civil engineer, capable of teaching civil engineering during the winter months."[19] Young is credited with several institutional buildings for colleges and seminaries, and the only major criticism he received prior to joining the Treasury Department was for over-using a Greek architectural style that was quickly becoming unfashionable[20]. His traditional, reserved style made him well suited for designing buildings for the federal government.

Treasury Department work: Young was appointed Supervising Architect of the Office of Construction in the Department of the Treasury from 1852 to 1862. Probably because of the extreme volume of work the Office of Construction was superentending and Bowman's disciplined military example, Young standardized much of the design process. Many of the buildings are structural copies of each other, only differing in the architectural style or details. At least four of the buildings were so identical that the same lithographing plates were used for printing the plans, with only the titles changed. As the major designier for the Treasury Department, Young carried out the majority of the technical communication with Cooper & Hewitt including this telling letter dated January 25, 1854[21]:

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[18]Overby, Osmund. "Young, Ammi B." *Macmillan Encyclopedia of Architects*, Vol. 4, p. 464. The Free Press, New York.

[19]Wodehouse, Lawrence. p 269.

[20]Overby, Osmund. "Young, Ammi B." p 463.

[21]From A.B. Young to Cooper & Hewitt, Jan 25, 1854; Letters Sent, Chiefly by the SA and the EIC Sept 1853-Aug 1852, Vol 1, p 277; Entry 6; Records of the Public Building Service, Record Group 121; National Archives, Washington, DC.

Gentlemen,

Will you have the kindness to send me a sketch of the cross section of your Rolled Wrought Iron Beams, shewing its accurate form and size, also inform me what is the greatest length they can be rolled, and the weight (distributed equally over their length) which they are calculated to sustain without any inconvenience or improper depression, on the longest length and other lengths decreasing by one foot each, together with any information in your possession connected with the subject, that may leade (sic) to a proper use of them, for building purposes by the Government.

I will also be much obliged to you, if you have any knowledge thereof, to inform me the size, shape, and cost of a large Tile recently designed to be used with the above beams for the construction of fireproof floors and from whom they can be obtained, To save postage to the Dep't, please enclose your answer in an Envelope directed to the Hon. James Guthrie Secretary of the Treasury,

Respectfully your ob't svt
Ammi B. Young
Sup'g Arch't T.Dept

There are several things to note here. The first of which is that he requested a cross sectional sketch, which would be necessary to determine the cross sectional area and moment of inertia. This is a pivotal point in time for theory concerning strengths of materials. Squire Whipple published the first edition of his landmark work, *An Elementary and Practical Treatise on Bridge Building* in 1847[22] which was highly influential. Herman Haupt, Thomas Tredgold, and Dennis Hart Mahan also published texts and papers on the subject at the same time. There were many sources that Young could have used to evaluate analytically the strength of the beams, but his background would have prompted him to rely almost exclusively on the experimental data available to him. There is the possibility that the West Point trained Capt. Bowman may have influenced his perceptions about engineering analysis.

[22]Whipple, Squire. *An Elementary and Practical Treatise on Bridge Building.*

By 1856, the staff at the Building Office included one full time computer/draftsman, so it is likely that some sort of numerical analysis was being performed. One must also note that the letter requests a table of lengths vs. strengths supporting a uniformly distributed load. Tredgold points out that his empirical results showed that a beam could hold twice the total load if it were distributed along its length, than if it were placed as a single point load at the center[23]. It is likely that this was recognized by Young and therefore he makes the distinction in the letter. (Today we know that the bending moment at the center of a beam for a point load is double the moment of the equivalent distributed load, but that was not widely understood at the time.) Still, in the end, one must come to the conclusion that Young, like many architects today, depended more on tabular information than any theoretical work for performing structural analysis. There are several letters to Cooper & Hewitt both by Bowman and Young concerning experimental tests to be performed, and the supervision of these tests. James Guthrie had $3,500 appropriated for testing on the Cooper & Hewitt beams. The 1854 series included tests on an 18 inch box girder, and several different arrangements of rail iron[24]. The tests detail load vs. deflection and permanent set is checked at most loading increments. The test results do not list the moment of inertia for the members, but dimensions are readily available, therefore it can be computed. The first test was performed on an 18 inch box girder, but the remainder are performed on samples of rail iron. The rail iron tests are performed in two series. First the rail is tested in several

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[23]Tredgold, Thomas. *Elementary Principles of Carpentry.* Printed for J. Taylor, London, 1820. p 45.

[24]Alexander, B.S. "Wrought Iron Beams" Ex. Doc. No 56., House of Representatives, 33rd Congress, 2nd Session.

configurations, including some built up sections, by itself. Later, the rail is tested as part of a segmental arch system, where three beams are used with two arch spans. The load is then applied at the center of the system. These were tested with and without the rail head since it was supposed that the rail head was not necessary because of the lateral bracing of the arches.

There are two main comparisons to be gained here. First, the tests obviously showed the necessity of the rail head, increasing the stiffness of the system by a factor of 1.5. Second, it is important to compare the stiffness of the beam with the stiffness of the beam/arch combination. The arches doubled the strength of the individual beam and increased its elastic stiffness by a factor of six. It is clear that the beams are not the only structural member in the system: the brick contributes substantially to its capacity. A further evidence of this is the load transfer to the side beams. At the terminal load the side beams showed approximately half the deflection of the center beam. Alexander reports that there was no visible cracking until the straps were loosened, at which time the center beam increased its deflection slowly from 1.125 inches to 4 inches. The beam showed substantial yeilding and gave at least some warning of failure. The test results were published for the Committee of Ways and Means, which was considering the appropriations for new custom houses and federal buildings[25].

Due to the significant benefits of a fireproof system and the apparant success of the testing, the appropriations for many of the buildings constructed during the late 1850's were

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[25]Alexander, B.S. "Wrought Iron Beams" Ex. Doc. No 56., House of Representatives, 33rd Congress, 2nd Session.

tied to the requirement that they be fireproofed with this particular beam and vault construction method[26]. The Secretary of the Treasury, James Guthrie seemed to have a penchant for encouraging new industries, so it is highly likely that the support shown by him and by Bowman for the fireproofing was an attempt to encourage the production of the new material. Indeed, the Department of the Treasury built no less than 16 buildings using iron from Cooper & Hewitt under the same appropriation alone. This was one of a series of windfalls for the struggling Cooper & Hewitt. As the only manufacturer of wrought iron I-sections, they became the building office's exclusive structural contractor through the rest of the 1850's. Although the corporation carried the name of Peter Cooper, the successful industrialist, its history shows other, equally dynamic players with futures of their own.

[26]Congressional Record, "An Act making Appropations for the Civil and Diplomatic Expenses of Government for the year ending the thirtieth of June, One thousand eight undred and fifty-five, and for other purposes." 23rd Congress, Session I. 1854. p. 571

Chapter 3: COOPER & HEWITT

The Manufacturer's Role in Design

Abram Hewitt

Abram Hewitt was born in 1822, the son of a highly skilled furniture maker. However, what his father possessed in talent, he lacked in business sense, and the family business fell on hard times during Abram's early childhood. Although the family resources were slim, Abram showed a prodigious skill for learning, devouring books as fast as he could get his hands on them. He earned a scholarships to Columbia College's grammar school and then later to Columbia College itself (which later became Columbia University). He quickly became not only an outstanding student, but a first rate tutor as well. His notes from the lectures were of such high quality, they are still on display at Columbia University as a prime example of note taking and course content during the 19th century. Hewitt lived with his parents only a few blocks from the campus, and again earned money from tutoring other students[1]. He graduated at the head of his class in 1842.

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[1]Nevins, Allen. *Abram S. Hewitt with come Account of Peter Cooper.* Harper & Brothers, New York, 1935.

Peter Cooper

Peter Cooper was one of many wealthy New Yorkers to hire Hewitt to tutor their children; in this case, his son Edward, who had fallen behind because of physical problems. The two families soon became close, and a life-long friendship developed between Abram and Edward. Indeed, later in life, Abram married Peter Cooper's daughter, Amelia, and the two families lived in a duplex house, even eating their meals together.

After graduation, Abram taught at Columbia College's grammar school for a year and practiced law, and then went on the traditional European tour with Edward. Abram wrote, in one of his letters to home, that he found in England, "more beauty and more deformity, more wealth and more poverty than my eyes have rested upon during the whole course of my life." The trip opened his eyes to the social and political conditions of the working class and probably greatly influenced him later as a politician in New York. Fortune seemed to follow the pair until the trip home, when they were shipwrecked. Facing death greatly matured both men and left a lasting impression on their lives[2].

In the meanwhile, prior to 1845, Peter Cooper had come to realize that without a cheaper source of raw materials his investment in his thriving rolling mill could be wiped out. This involved moving the mill to a site near the sources of pig-iron and anthracite coal to reduce transportation

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[2]ibid. pp 30-44.

costs and speed up production. The most attractive site seemed to be Trenton, NJ, on the Delaware river. Water power was available from the Trenton Waterpower Company, of which Peter Cooper later bought a 90% interest. Rail transport from the factory used the Camden & Amboy rail line that ran from Philadelphia to New York through Trenton. For water transport, the Delaware Canal as well as two other canals, the Morris and the Delaware & Raritan, assured easy transit from the factory to the interior of the country. In addition to these valuable transportation routes, labor was abundant and land was relatively cheap for both plant needs and local company housing[3].

Certainly, such a large operation would be difficult to manage from New York. Markets would need developing, customers would need to be found, and a quick, effective decision maker at the plant would be needed to assure its success. After the European tour Hewitt was nearly penniless and needed to start work at something almost immediately.

Peter Cooper felt that Edward was now ready to start serious work and proposed to put him in charge of the new works at Trenton. To the elder Cooper this seemed ideal. Edward had "conspicuous talents of an inventive and mechanical nature" as well as "all his father's geniality, kindness, and bluff ability

Edward Cooper

[3]ibid. pp 82-85.

to extract loyal effort from employees."[4] Edward, however, felt lacking in business abilities and insisted that Abram join him. Stories were frequently told of how many telegraph blanks Edward would tear up before making a decision. Abram, on the other hand, was resourceful, decisive, and determined. Peter Cooper had a fear of partnerships and at first refused. Eventually, the elder Cooper allowed them to form their own corporation in which he would put up the majority of the starting capital. Some credit could be secured and Peter Cooper would incur only limited liability. Edward would superintend the plant and all mechanical operations, and Hewitt would manage the business. Peter Cooper's name carried much weight and gave the business instant credibility. This credibility was further enhanced by the policy paying all obligations promptly at maturity and paying its workers in cash. Although they did have a company store that sold goods at cost on "Helper's Row", the workers were not necessarily tied to it. In a time of ubiquitous paternalism, Trenton Iron went out of its way to instill a sense of personal responsibility in its workers: responsibility not only for themselves, but also for the company and its product. From the beginning of operations Edward Cooper and Hewitt felt a "keen mortification" when any rails were rejected as crooked or brittle. In 1848, a special employee was hired to record the quality of the work done by the puddlers with an eye to correcting any problems. Not only were the number of bad balls (blooms) recorded but the reason for the problem so that they could be corrected[5]. By 1850 a system of profit sharing was put in place to assure the workers "a more immediate interest in the profits of the business."[6] Even by

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[4]ibid. p 82.

[5]ibid. pp 89-90.

[6]ibid. p. 89. From the Director's minutes, kept by Abram Hewitt, as Secretary, currently in the Hewitt papers.

today's standards, this was a very enlightened corporate position.

In 1847 the company was incorporated under the name of the Trenton Iron Company. Of the \$300,000 in shares, Peter Cooper bought \$151,000 and \$149,000 was obtained by Cooper & Hewitt, a managing company set up by Edward and Abram. To pay for their shares Cooper & Hewitt borrowed \$50,000 from a local bank. The bank secretly asked Peter Cooper for a guarantee on the loan, which he provided without telling them, which was against one of Peter Cooper's main rules: "Never endorse anybody's note; *give* a man money, if you want to, but don't endorse his paper." Peter Cooper was designated President of this new company, but Hewitt and Edward Cooper were the managers of the company under Cooper & Hewitt. This avoided the partnership that Peter disliked and was a distinct advantage to Trenton Iron. Cooper & Hewitt charged \$25,000 per year for their managing services which was much cheaper than the ordinary commissions that would have been paid to a discounter[7]. Much later, when Trenton Iron was struggling, Cooper & Hewitt was dissolved to eliminate this cost to the company, although Abram continued to perform the same management functions.

[7]ibid. pp.91-93.

The Product

While Edward Cooper was busy setting up the mills, Peter Cooper and Hewitt were busy obtaining orders for iron. Initially the complex consisted of a wire/rod mill and a rolling mill. Pig iron and coal were initially bought and delivered to the mills through the canals and railways. By the mid 1850's Trenton Iron was involved in all phases of the iron production from coal and iron mining to final product. One mine was especially pivotal to the company's success. Hewitt purchased the Andover mine for $9,500 in 1847, a "steal" for a mine with abundant reserves and high quality ore. The mine was a source for both red hematite and blue magnetite that when mixed produced an unusually tough wrought iron[7]. This was important since American rails had a reputation for brittleness. There were numerous stories of how the rail heads would eventually shear free from the web and curl up, frequently with disastrous consequences. In early 1846, Robert and Edwin Stevens had become dissatisfied with the quality of the British rail they were using and contracted with Trenton Iron to buy rail for the Camden & Amboy railroad at the same price as the British rails: $90 per ton. This started a profitable long term relationship between the powerful Camden & Amboy and Trenton Iron, which was later augmented by numerous other rail companies[8].

Hewitt learned early that production diversity assured the long term stability of the business. Early on, transferring production emphasis to the wire plant saved the business when rail iron could not be produced in the US as cheaply as in Britain. In the process, Hewitt

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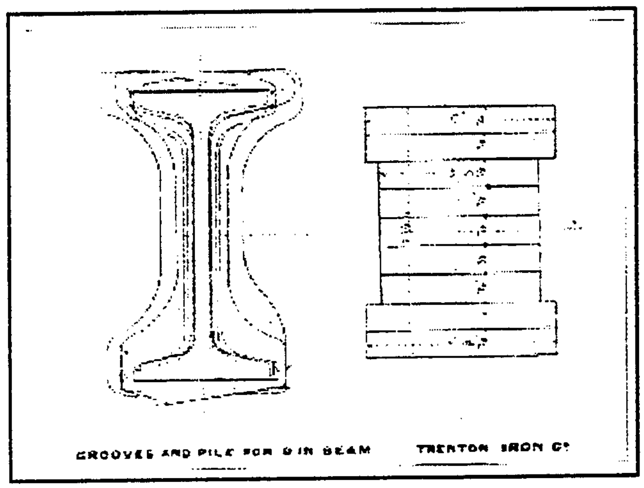
[7]ibid. pp 90-91.

[8]ibid. pp 86-87.

convinced John Roebling to move his operation to Trenton[9]. Roebling bought only Trenton Iron wire and bar stock for his cables for many years. In the mean time, to reduce production cost on the rails, Hewitt surfaced them with Andover iron, heat treated them, and hoped to rely on the quality of the product for orders[10]. The rails did eventually become competitive again and the reputation of the rails won him contracts with not only the Camden and Amboy, but most of the large rail companies in the Northeast. When a glut of rails again flooded the market in 1854 because of increased production from the iron ore mines in Britain, Trenton Iron was already in the process of creating a universal mill that could roll I-sections to be used in bridge and building construction as a fireproof system. William Barrow, a capable English inventor started the design of the rolling mill but died before it was completed. Charles Hewitt, Abram's brother, finished the design, but not before spending $150,000 on its development. The mill was essentially the first universal rolling mill, a mill that could produce many different sizes and sections[11]. The drawings for the mill indicate that several iron bars were stacked on top of each other and then rolled through the mill at an elevated temperature to weld the pieces together and



Groves and Pile for I-beam

[9]ibid. p 105

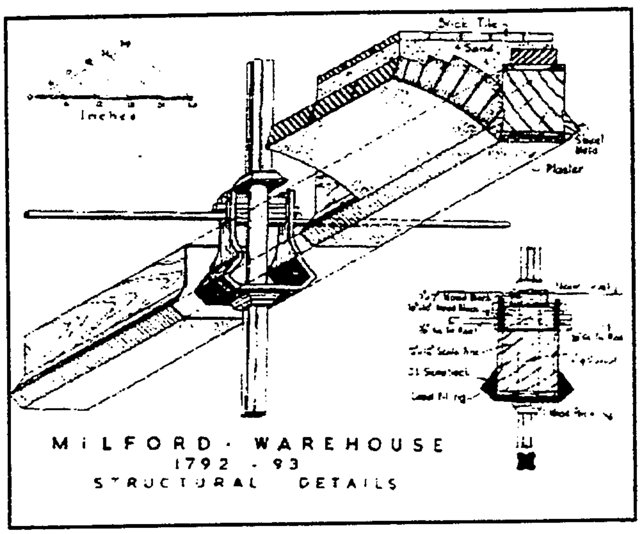
[10]ibid. It is unknown how this was accomplished, but was one of the first evidences of this type of process. It is possible that only the rail heads were made of Andover ore, but it is difficult to determine whether this is the case.

[11]ibid. p 114.

shape them into the I section. Unfortunately, it appears that they did not use a high enough temperature to produce uniform material properties.

By 1853, beams were being produced that were 7 inches in depth and 25 feet long. Cooper & Hewitt were under orders from the Treasury Department to ship the deepest sections that they could produce and by 1856 when the iron was shipped for the Wheeling Custom House, the I-sections were at least 9 inches in depth.[12]

Fireproof Construction



MILFORD · WAREHOUSE
1792 - 93
STRUCTURAL DETAILS

The segmental arch floor system dates from 1792 in William Strutt's Millford Warehouse in Derby, England[13]. Originally, in this type of system, the supporting beams were heavy timbers with an additional layer of plaster underneath, which would generally only char in a fire, leaving the floor system intact. Sand was used over the arches and surfaced with brick tile to protect the system from the top.

The buildings constructed by the Treasury Department used the new wrought iron I-beams instead, with the bottom flange of the beam supporting the brick arches. Wrought iron box

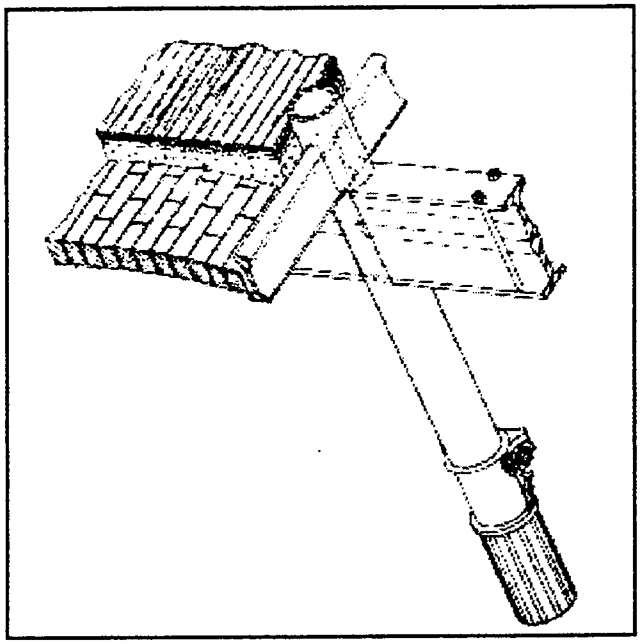
[12]From Young, 1856. "Letters sent, chiefly by the SA and the EIC" Sept 1852-Aug 1862. Records of the Public Buildings Service, Record Group 121. The National Archives, Washington, DC.

[13]Peterson, Charles E. ed. *Building Early America*. Chilton Book Company, Radnor, PA. 1976. pp 100-1.

beams then supported the iron joists, transferring the load to cast iron columns on each floor that are connected continuously from floor to floor.

The final step in the evolution of the arch system was to replace the brick with hollow clay tiles which were lighter and even more fire resistant. Young requested information from Cooper & Hewitt on clay tile for use in this system, but there is no record of their use in any of the buildings constructed by the Treasury[14].

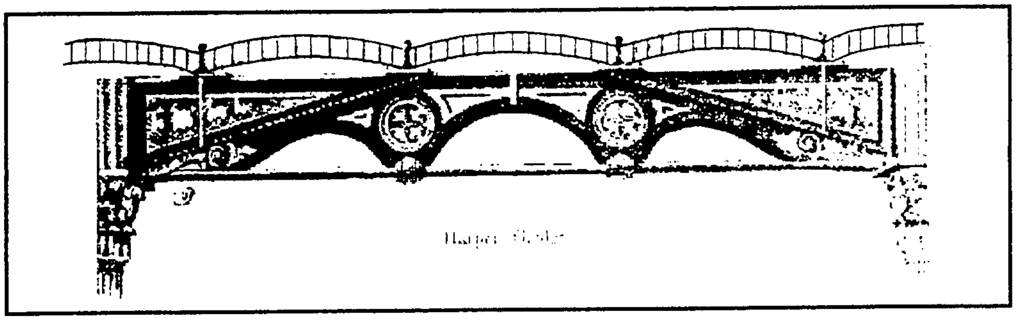
Cooper & Hewitt's aggressive marketing tactics assured that this system was extensively used during the 1850's and 1860's. Whenever a building was destroyed by fire, Hewitt made sure the owners were told of the benefits of his "fireproof" system. The Cooper Union building was the first to recieve these beams and utilize the beam and vault construction later seen in the Wheeling Custom House. The first two floors were completed at the Cooper Union in 1853, and then production shifted to providing beams for the rebuilding of the Harper & Brothers printing press building which had been destroyed in a fire. The building covered more than half an acre and was seven stories high. J. Henry Harper wrote later that the building was used until after the First World War by the publishing house and gave



Cooper Union Framing

[14]From Young to Cooper & Hewitt, Jan 25, 1854. "Letters sent, chiefly by the SA and the EIC" Sept 1852-Aug 1862. Records of the Public Buildings Service, Record Group 121. The National Archives, Washington, DC.

"the completest satisfaction."[15]



Harper Building Framing

Trenton Iron at that point was commencing testing on the beams under the aegis of the government and the favorable results quickly produced a large demand for these beams at twice the price of rails. The government itself ordered 6000 tons of iron which was rolled during the winter of 1855-1856. Nevins reports that "nearly every important public edifice constructed in America in the five years after 1854 was equipped with Trenton iron."[16] Because the system was believed to be fireproof, many insurance companies lowered their rates for this type of construction[17] and corporations were quick to try the new system. There are numerous examples still in use in cities like Charleston S.C, Seattle, WA, and Washington, D.C., including the Capitol extention built by Montgomery Meigs. These were the structural antecedants to the skeleton frame used so extensively in the Chicago school of architecture.

The brick, in theory, would contain any fire to a single floor. The only flammable materials used in the interior of the building were the pine flooring, sash, and the interior doors.

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[15]J. Henry Harper. *The House of Harper*, 104-106. Quoted in *Abram Hewitt*, p. 116.

[16]Nevins, Allan. *Abram Hewitt*. Harper & Brothers, New York, 1935. p. 117.

[17]C&H Book No. 17, New York, Jan 3, 1855. pp 256-258

Windows were equipped with cast iron shutters that were to be closed during a fire. Since there were no exposed combustables, the system was thought to be virtually fireproof. The Chicago fire in the 1870's proved it to be vulnerable due to the collapse of the iron structural system in the heat, destroying the Custom House there as well as many other similar buildings. Although the iron does not burn, it looses stiffness at temperatures much lower than its melting point (800-900°F). Without the stiff iron supports for the brick vaulting, the floor system collpases. This type of collapse had been seen previously in Europe by Fairbairn and others, but the failure was credited to a column collapse, rather than member collapse[18]. In 1854 Fairbairn published a text entitled *On the Application of Cast and Wrought Iron to Building Purposes.* There is evidence to suggest that Cooper & Hewitt had access to this book even though the date of publication is 1854, the same time period in which this system began to be advertised by Cooper & Hewitt. The timing seems to be more than coincidental. It is likely that Abrahm and Edward saw this type of building practice while on their tour of Europe, and its initial use in the Cooper Union building may have sprung from that experience. However, Bowman's correspondence files carry several letters referring to tests to be performed on a similar system that replaces the brick arches with iron plate arches[19]. This type of construction system is discussed in detail in Fairbairn's book. This interest may point to the influence of that text on either Bowman or Hewitt, although the latter may be more likely. In addition, the section on Fireproof Warehouses seems to be closely

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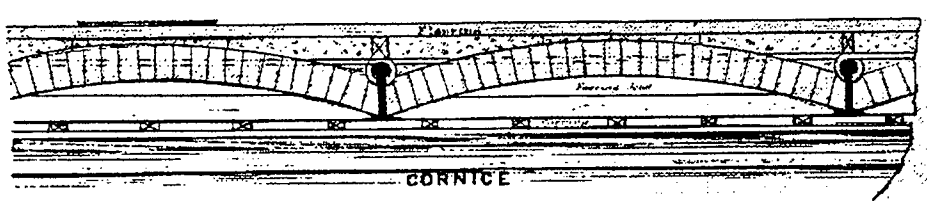
[18]Fairbairn, William. *On the Application of Cast and Wrought Iron to Building Purposes.* pp 118, 130.

[19]From Bowman, 1854-5. Vol 1, p. 354. "Letters sent, chiefly by the SA and the EIC" Sept 1852-Aug 1862. Records of the Public Buildings Service, Record Group 121. The National Archives, Washington, DC.

adhered to in the design of the custom houses. Fairbairn recommends a rise to span ratio of 1/10th, which is followed by the designs. He also recommends that the tie rods be placed as far down as possible, but recognizes that tie rods on the bottom flanges would give the structure an "appearance of complication," and therefore recommends that they be imbedded in the arches to conceal them from view.[20] The iron I-beams in the Custom House all have holes in the center of the web to carry similar tie rods. However, the specification drawings show tie rods looped over the heads of rails (see illustration below)[21]. This is contrary to what is recommended in Fairbairn's work and obviously not as the system was actually constructed. It is possible that the changes in the design were made after the specifications were drawn, especially in light of the repettitive nature of the designs eminating from the treasury department. It may have been easier administratively to keep the specification drawings the same even after the I-section was substituted for the rails. Fairbairn further recommends that a bond rod, another iron beam, be embedded in the side walls to solidify the construction, as is seen in the Treasury Department designs.



Detail of floor system specification
for the Wheeling Custom House

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[20]Fairbairn, Sir William. *On the Application of Cast and Wrought Iron to Building Purposes.* p. 135

[21]Bowman, A. *Plans of Public Buildings in the Course of Construction Under the Direction of the Secretary of the Treasury including the Specifications Thereof.* "Wheeling, Virginia" A copy is on file in both the Custom house in Wheeling and at the National Archives in College Park as part of their collection of oversized materials.

The appendix of Fairbairn's text contains two papers concerning the failure of a cotton mill in Oldham, England in 1844. One is a report on the failure itself and the other a report to The Institution of Civil Engineers, given in April of 1847, entitled "On some Defects in the Principle and Construction of Fire-proof Buildings." The publication dates of these papers grants more time for the information actually to reach Cooper & Hewitt. In these papers, Fairbairn strictly warns against designing without thorough information as to the strength of the members involved, which may have provided a strong motive for the testing Bowman supervised.

Although there is only circumstantial evidence to suggest that Hewitt had access to these resources, it is clear that he did have access to Fairbairn's *An Account of the Construction of the Britannia and Conway Tubular Bridges*[22] In a broadside distributed by Cooper & Hewitt promoting the use of the new iron beams, Hewitt makes the remark that the girders are constructed "on the most approved philosophical principles developed in the thorough experiments of Messrs. Stephenson, Hodgkinson, and Fairbairn, made in reference to the construction of the Great Tubular Bridge over the Menai Straits."[23] This text details a series of tests on tubular girder bridges and was widely studied by engineering practitioners of the day. Theoretically the major contribution of this work was to widely distribute information on the correct placement of the neutral axis[24]. Thus bending stresses could be accurately computed, although plastic behavior is not clearly addressed. Since stress limits were around 1/3 of the

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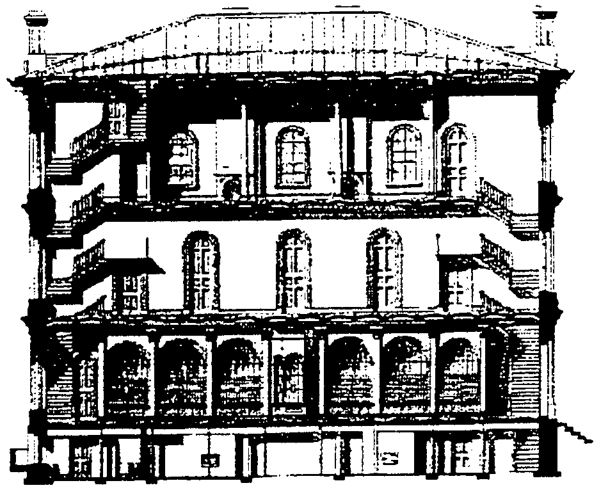
[22]Fairbairn, Sir William. Weale & Longman, London. 1849.

[23]Bowman's Annual Report, , Sept. 1, 1854; Military Records, Record Group 77; National Archives, Washington, DC.

[24]Timoshenko, Stephen. *History of Strength of Materials.* Dover Publications, Inc. New York, 1983. p.127

ultimate strength, material behavior remained elastic and plastic behavior was not a serious concern.

Chapter 4: Building History



Although the design for the Custom Houses constructed by the Treasury has been examined in general, it may be more important to understand the details of the construction, use, and alterations that pertain to the structure of this particular Custom House in Wheeling. The purpose of this chapter is to examine these details and provide insight into the context for its construction and use.

West Virginia Independence Hall, in Wheeling, West Virginia, was completed on April 7, 1859 by the Department of the Treasury to house the Customs service, a Post office, and a federal courthouse. The building was originally designed as a three story building with a basement, and measures 85 ft. by 60 ft. Architecturally, the style follows the Italian Renaissance *pallazo*, or palace, with Greek details. The columns closely follow the Greek orders from Doric in the basement through Corinthian in the Court house on the third floor. Engaged columns were rendered as square pilasters, but carry similar capitals. The details are carefully executed according to the classical forms and convey a sense of formality appropriate for the local symbol of the federal government. The first story housed the post office for the City of Wheeling. The second story held the offices for the US

Custom service and the third held a federal court house. The site was situated halfway between the Hempfield and B&O Rail Road Depots, and was noted to be in a growing area of town[1]. Indeed, Wheeling was just one example of the growing commercial enterprise that came with the improved passages on the Mississippi, Missouri, and Ohio Rivers that had been gained in the first half of the 19th century.

A Context of Expanding Trade

By the 1850's Wheeling was fast becoming a transportation hub and was experiencing a construction boom. From the end of the War of 1812 both national and international trade had been increasing at a rapid pace. The purchase of the Louisiana Territory opened up vast new areas for settlement, as well as access to all of the Mississippi, Missouri, and Ohio rivers. This meant that steamboats could transport goods as far as Wheeling, which was the head of summer navigation on the Ohio River. Because of this, in 1831 Wheeling was designated a Port of Entry[2]. Earlier, in 1818, the National road reached Wheeling from Cumberland, making travel easier from the east[3]. The Wheeling Suspension bridge, extending the national road over the Ohio River, was completed in 1849 and the Baltimore & Ohio Railroad line to Wheeling was opened in January of 1853[4]. The railroad brought supplies for extending the National Road into Ohio and picked up goods that had been shipped on the Ohio River. Ship repair and construction were major local industries, as well as cast and wrought iron production, nail making, and glass production.

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[1]Written report submitted by A.J.Pannell and George Cracraft to James Guthrie, September 14, 1854. WVIH Collection.

[2]Marshall, Paul and Marshall Frances. Historic Structure Report, p 18.

[3]Searight, Thomas B. *The Old Pike: A History of the National Road.* Uniontown, PA, 1894. p 16.

[4]Jacobs, Timothy, ed. *The History of the Baltimore & Ohio.* Crescent Books, NY, 1989. p 29.

The request and preliminaries

The customs officials had rented some offices in Wheeling, which were soon too small to handle such a growing transportation center. It was clear that a new customs office was needed. In addition, being at the head of navigation, it was important to have a large post office and federal court. In December of 1850, a petition was circulated asking Congress to appropriate funds for a "proper building in Wheeling for a Custom House, Post Office, and other offices..."[5]. This need was felt by more than just the local community. In May of 1853, Andrew Jackson Pannell, the Surveyor of Customs for Wheeling, wrote to James Guthrie echoing the community's requests:

> "By recent instructions received from the Secretary of Treasury, it appears that the local inspectors should apply to the Office of Customs for a place within the custom house to transact their business in. There being no custom house here at present I am at a loss how to proceed in their case. They have at present rented an office at the rate of $50. per year on their own and will the department allow it.
>
> It seems to me to be necessary for the protection of books, papers and funds belonging to the government that here should be a safe attached to this office for the better protection of them against fire. On the night previous to my coming into office it caught fire and was very near being burnt down.[6]"

Legislation was submitted by Dr. Kidwell to appropriate $88,000 for the construction of a federal building in Wheeling and was approved in the 23rd session of Congress as part of a group of 16 custom houses and marine hospitals to be constructed by the Department of the Treasury[7].

[5]Wheeling Daily Times & Gazette, December 20,1850. WVIH Collection

[6]Microcopy 174, #49, 1853, p 300. National Archives, Washington, DC. Originally cited in Marshall et al. *Historic Structure Report*

[7]Congressional Record, "An Act making Appropriations for the Civil and Diplomatic Expenses of Government for the year ending the thirtieth of June, One thousand eight hundred and fifty-five, and for other purposes." 33rd Congress, Session I. 1854. p. 571

Capt. Bowman paid a visit to Wheeling to select the site in November of 1854[8]. The site was selected largely because of its proximity to both the downtown area and the port. Its description indicates that it is above the flood plain, but a letter sent to Bowman in December indicates that the site had been flooded to a depth of over three feet in 1832 and 1852[9]. In a letter to Guthrie in March of 1855, Bowman concludes that although the site had been flooded twice in the past 25 years, "no great amount of inconvenience" could have arisen since most of the "most prominent citizens" could not even remember it had been flooded. He states, "I do not see any objection of sufficient weight to counterbalance the decided advantages, that the lot recommended posesses over the others.[10]" Although the site was prone to frequent flooding throughout most of its life, there have been no recorded problems with settlement.

Initial Construction and Early Use

Construction contracts were signed in June of 1856 and the construction was completed in April of 1859, ten months more than had been estimated. The superintendent of construction, James Luke, wrote Bowman once complaining that the stone was of low grade, but otherwise there were few problems except for foul weather[11]. The first of many maintenance problems occurred only 5 months after opening. Rainwater was draining into the basement, and this was corrected by raising the sidewalk so that the water would drain into the street instead, at a cost of $84.85.[12]

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[8]The Wheeling Intelligencer, November 12, 1854. WVIH Collection

[9]Letter from Mr. Clemens, Dec 28, 1854.WVIH Collection.

[10]Letter to J. Guthrie, Sept 1854, WVIH Collection

[11]Monthly report for April 1857, letter to J. Guthrie, May 1, 1857. WVIH collection.

[12]Letter from A.J. Pannell to Guthrie, Sept 12, 1859. ibid.

During the Civil War the building was used to store munitions for the troops. This is highly significant to this study in that the probable load from boxes of guns is between 100 and 150 psf, stacked 4 to 6 boxes high. It is not likely that the munitions would have been stored in the basement, as it was frequently prone to flooding which would have ruined the supplies, (which at one point included a small amount of gunpowder). Although this load is well beyond the current rating of the structural system, the floor system shows no major sign of distress. In addition, the structure was also used to house the West Virginia State Library, taking up two rooms on the second floor.[13] This load type is also much heavier than its current rating, indicating the possibility of additional capacity.

Renovations

The first renovation occurred in 1867 and included major roof repairs. The original iron roof had been severely corroded by sulfur smog from the iron works in the area, and was replaced with a slate roof. In addition, the roof pitch was raised to a top height of 6 ft. for the slate, and the gutters repaired. Inside, additional shelving was added to accommodate the steamboat inspector's files.

The next major renovation was done in 1874. A set of stairs that spanned from the basement to the back of the first floor post office was moved over to the right side of the building to accommodate more space for the post office. A steam heating system was also installed.

There had been constant complaints about the original heating system in the building which was modeled after a French "background" heating system[14]. The hollow, cast-iron columns in the building were used to transfer heat from the furnace in the basement to the floors above, and also acted as an additional fireproofing precaution because the moving air in the columns would help

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[13]Conversation with Custom House researcher, Beverly Fluty.

[14] Peterson. *Building Early America*. Chilton Book Company, Radnor PA. 1976.

maintain a lower temperature in the case of a fire[15]. This heating system was only designed to bring the temperature up to around 50°F, therefore additional fireplaces in each room were used to bring the temperature up to a normal range.

The prevailing theories about heating and ventilation seemed to favor this type of arrangement. At the time, air that had been expelled by a person or that came from a fire was termed "vitiated" and was considered unhealthy to breathe[16]. With the background heating system, the fire in each room could be smaller than otherwise necessary reducing the amount of "phlogisticated" air being introduced into the room. This was never completely understood, and the individual fireplaces were apparantly inadequate for the comfort of the workers because they requested stoves to replace the fireplaces, and by the time of the 1867 renovation, the furnace had been out of use for six or seven years[17].

The most controversial renovation was completed in 1888. Against the recommendation of both superintendents of construction, the firm of McCarthy & Baldwin was contracted to build a new set of stairs and an elevator on the right side of the building. A semi-circular addition was added on the end to accomodate the spiral staircase and an elevator. Much conflict arose between the construction superintendent and the contractor. This was due in part to alterations in the plans that had occurred in Washington. The result of these changes was that the plans and specifications did not agree in many major details. The stairs were completed, but instead of allowing a four foot wide area in the center to accomodate an elevator, only a two foot space was left, therefore the

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[15]This was initially described by Fairbairn, in *On the application of cast and wrought iron to building purposes.*

[16]Peterson, Charles, ed. *Building Early America.* Chilton Book Company, Radnor PA. 1976. p174.

[17]Marshall, Paul and Marshall, Frances. WVIH/WCH Historic Structure Report, p 36.

construction of the elevator was abandoned. The addition was very unpopular with the town and construction was greeted with several scathing editorials[18]. Architecturally, the addition was inconsistant with the rest of the building, and cost overruns were numerous[19].

Private Ownership

In 1908, the Old Post Office Improvement Company bought the building from the federal government, in hopes of restoring it, but sold it four years later to the Conservative Life Insurance Company, who used it for their offices and rented out space on at least the first floor to a bank and several other merchants[20]. While Conservative Life owned the building, they added an additional floor and two additional bays, covering up the semi-circular addition. Little detailed information is available about alterations and use during this period, but it is likely that some of the rooms in the upper floors were used for files, which would be likely to load the floor much more heavily than current analysis would allow. For several years, the first floor was leased to a bank and a vault was constructed on the entrance floor. To carry the weight of the vault, an area underneath it was filled with sand and walled in so that the sand would suppport the floor system[21]. It is questionable as to how much load the sand would have carried, and likely that the floor itself was forced to support the majority of the weight of the safe. In 1951 the building was again sold to the American National Insurance Company, who sold it to the Pythian Building Corporation in 1963.

The State of West Virginia bought it as a state landmark that year, opening it for the state's

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[18] Spence, R.Y. *The Wheeling Intelligencer*. Nov 4, 1889.

[19] ibid. pp 53-63.

[20] ibid.

[21] Conversation with Researcher, Beverly Fluty

100th birthday, and has owned, operated, and restored it since then. During its ownership, the state has performed several major renovations to the building, in an effort to return it to its condition in 1863, when West Virgina became a state. The additional bays and floor have been removed, and the heating and air conditioning system installed. In addition, the roofing system has been redone and light fixtures have been made to match the original gas light fixtures for the building[22].

There have been numerous suprises during renovations. Restoring the frescos in the courthouse uncovered animal grafitti placed there by the original workmen which was later covered over by the painting and fresco work. A portion of the courthouse wall has remained uncovered to display the entertaining sketches[23]. Another, less pleasant, suprise occurred when remodeling the heating and ventilation uncovered several iron beams that had been cut to allow ductwork to pass under doorways[24]. This seriously jepordized the shear capacity of the connections and was quickly corrected. Another, more recent, suprise was the discovery of numerous casting cracks in the basement columns. Some of these cracks are merely vertical cracks that are typical of that period, but at least one column shows diagonal cracking that is much more severe. The cracks frequently extend into the new concrete footings, demonstrating that there is at least some movement in the crack, but the size of the footing crack seems to indicate that it may stem from temperature variation which has been greatly reduced in the last few decades because of climate controls installed in the most recent renovations. There appears to be no substantial amount of cracking in the brickwork that forms the floor system, but microcracks are entirely possible.

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[22] Marshall, Paul and Marshall, Frances, WVIH/WCH Historic Structure Report, Appendix D.

[23] ibid

[24] ibid

Currently the building functions as a museum, although the master plan recommends several other commercial and community uses for the building. The corridors of the building are currently rated for 125 psf, which is more than adequate for corridor and assembly loads. The side rooms have a longer span and therefore are only rated for 75 psf, which is adequate for museum and office occupancy, but inadequate for assembly purposes. The original customs office on the second floor is a long room with high ceilings and it was recommended in the initial master plan that this room should be used for as the main assembly area and loaned or rented out for community events and gatherings or commercial meetings[25]. Unfortunately it is one of these side rooms and fails to meet building code requirements for assembly loads (100 psf). The third floor court room has fixed seating arrangements on the outside bays that prevent overloading, and the main court area us fully capable of carrying assembly type loads. Currently the museum is using the basement as a small theater partially because other areas in the building are inadequate for that purpose. The architectural details of the building are consistant with formal uses, such as meeting areas and professional offices, and could be used for many state functions if it were not for its inadequate structural capacity. If past performance can be used as a guide, the numerous overloading cases seem to argue for a much stronger system than the previous calculations would lead one to believe.

[25]Wrenn, Tony. Master Plan, ibid, appendix E

Chapter 5: Material Properties

Once the potential loads on a system are determined, it is important to establish the material properties of the components with some accuracy. In a typical modern design or analysis the material properties are specified by the engineer, and tested to confirm their adequacy. In an overall sense, the material properties are assured by the manufacturer and governing boards that exist in the engineering profession. The engineer can be confident that the material properties given are not only correct, but uniform across both time and locality. Reasonable material variability is then accounted for through statistical process control and either statistical design methodologies or "safety factor" methodologies.

With historical materials this type of information may not be readily available. Even if material properties are available for the system in question, the age of the material may compromise that data. In an ideal situation, material taken from the existing structure in an inconspicuous area is used for proof testing. The results from destructive testing are generally seen as very reliable for material property determination if the tests are performed with skill and care. Typically with metals, a tensile test is performed. The most reliable tests are performed on cylindrical specimens that look similar to a dumbell (Fig 1). This provides a larger area for the machine to grip the specimen and hopefully assures that the yielding and failure will occur in

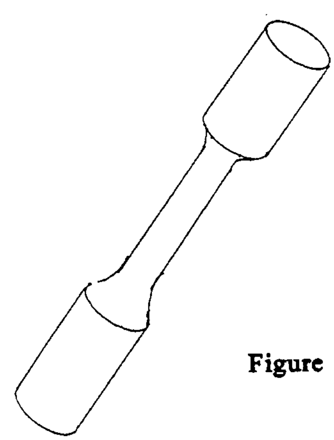
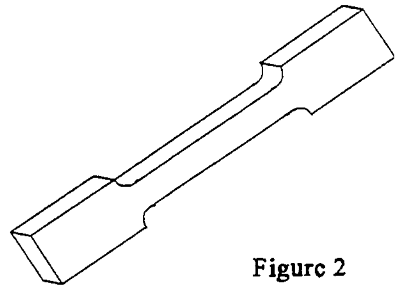


Figure 1

the center of the specimen instead of the ends. The cross sectional area can be accurately computed for the gauge area. Circular specimens are much more difficult to fabricate. Therefore a flat "dog bone" specimen is also frequently used (see Fig. 2). In both cases, the specimen is pulled at a uniform loading rate. Elongation or strain is measured by either a clip gauge or an electronic strain gauge and subsequently plotted against the stress in the gauge area. This provides information about the stiffness of the material and its yield characteristics. A similar process is used to test the compressive properties of a material, typically with smaller specimens.



Figurc 2

Unfortunately, destructive testing is not always possible or even desirable. Destructive testing is not only expensive but removing part of the structure may be hazardous or may adversely affect the historic fabric of the artifact. There are numerous non-destructive testing techniques that can be used to find the type of information required to establish the current capacity of the structure. In the future, non-destructive testing may even replace normal destructive testing techniques, especially for existing structures. Currently, NDT techniques are still under serious scrutiny because of the lack of experience associated with the methodologies.

Therefore, it is important to know the limitations of any testing technique. One way to explore the usefulness of a NDT technique is to study the affects of the variability of the individual parameters on the results. If the uncertainty of measured variable causes a range of results that are not sensitive enough to provide the information needed, a different test may be in order. This kind of parametric study can be of immense value in determining the effectiveness of a particular test and can prevent wasting time and energy on tests that cannot give useful information.

It is also important to remember that although quantitative information is important, qualitative information can be just as important. For example, in the case of this custom house, cracks were found in the basement columns. Infrared cameras were used to identify the extent of the cracking in a qualitative sense, and determine that the cracking did not extend throughout the entire length of the member, although it does extend through the thickness of the section. This testing brought us to the conclusion that the cracks were casting defects and not caused by overloading in the system. It also demonstrated that the cracks that extend from the column to the footing are probably due to thermal expansion and contraction rather than movement due to excessive loading.

The iron from the I-beams found in the Custom House in Wheeling have been tested through both destructive and non-destructive testing techniques. In part, this allows for validation of the NDT technique, but also provides additional information about the members themselves. Historic resources have also been of significant help in determining how manufacturing processes and testing procedures have affected the structural capacity of the members.

Brick properties.

Before proceeding, it is important to point out that the majority of the effort in determining the nature of the materials in this system have focused on the wrought iron instead of the brickwork. This is for several reasons. First of all, due to the construction method and the system configuration it is likely that the iron will control the capacity of the system. In addition, there was a serious concern that the iron would be extremely brittle and would therefore require much more stringent controls because of the potential for catastrophic collapse. Iron samples have been readily available for testing in a number of ways, while brick samples have been much more difficult to obtain and verify as original. Several areas of the exterior walls have undergone major structural changes including the addition of new brickwork on the interior. Further, while there are many widely used testing techniques for ferrous metals, testing existing brickwork either in-situ or in the lab is expensive and the techniques are not well known or well validated. There are numerous tests that can be performed on the constituents of new masonry work, but testing existing masonry is a relatively new field in the US. Although flat jack tests have been developed in Europe and are beginning to be seen here, they would be inappropriate for the type of arch construction we are considering in this case. There are several similar testing methods that have been certified and used consistently in Europe, however the US has been slow to keep pace with this field and few ASTM specifications have been established for such methods. Brick is rarely used as a structural material in the US, and the certification process for testing methodologies is lengthy.

The specifications requested hard burnt brick and there is no note of any trouble procuring adequate brick in the initial construction reports. The brickwork itself is remarkably even in color indicating a high degree of uniformity in the quality of the brick. Four bricks were selected from the site and tested in compression using a Celotex capping material on half brick samples. Upon slicing the brick open, samples 1 and 2 were determined to be extruded brick from a factory process and probably date from this century. They were tested and showed an average compressive strength of 4100 psi with a standard deviation of 150 psi. The failure mode for these samples was in both split tensile and shear. Samples 3 and 4 were hand-made and probably date to the original construction. Their average compressive strength was 6050 psi and had a standard deviation of 1500psi. Adjusting for the capping material, the corresponding compressive strength is 6765 psi[1]. Poisson's ratio for brick ranges from 0.04 to 0.11[2] and the higher value will be assumed for the prism, partly because the mortar will undoubtedly carry a higher Poisson's ratio. The typical failure mode for brick prisms is a split tensile failure because of the poor match between the constituents stiffnesss and Poisson's ratio. Since the mortar shows much more transverse elongation under load, this sets up a tension field in the brick which eventually causes spalling and tensile failure of the assemblage[3].

The mortar used is a natural cement lime mortar that is plentiful in western Pennsylvania and the Ohio Vally area. According to the Structural Clay Products Institute, natural cement lime mortar exhibits properties similar to type N mortar[4]. According to the 1992 ACI Masonry Specification, these properties give a prism strength (fm) of 1830 psi[5]. The mortar appears in good shape although the joints are not as carefully tooled as one would expect on visible exterior masonry work. This is not surprising since the majority of the brickwork was covered by plaster at the time of the initial construction and the main purpose of careful tooling is to reduce water migration which is not a problem on a structure's interior. Cracking is limited by the deflection limitations of the system and no visible cracks are evident now.

Any additional study of the strength of this system should include more extensive testing to

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[1]Monk, C.B. "Research Report Number 12, A Historical Survey and Analysis of the Compressive Strength of Brick Masonry." Structural Clay Products Research Foundation, Geneva, Il. July 1967. Table 2-1, p 6.

[2]Schneider, Robert R., and Dickey, Walter L. *Reinforced Masonry Design* Prentice Hall, Englewood Cliffs, NJ. 1980. p 45.

[3]*Monk, C.B. p 5-7.*

[4]*ibid. pp 8,9.*

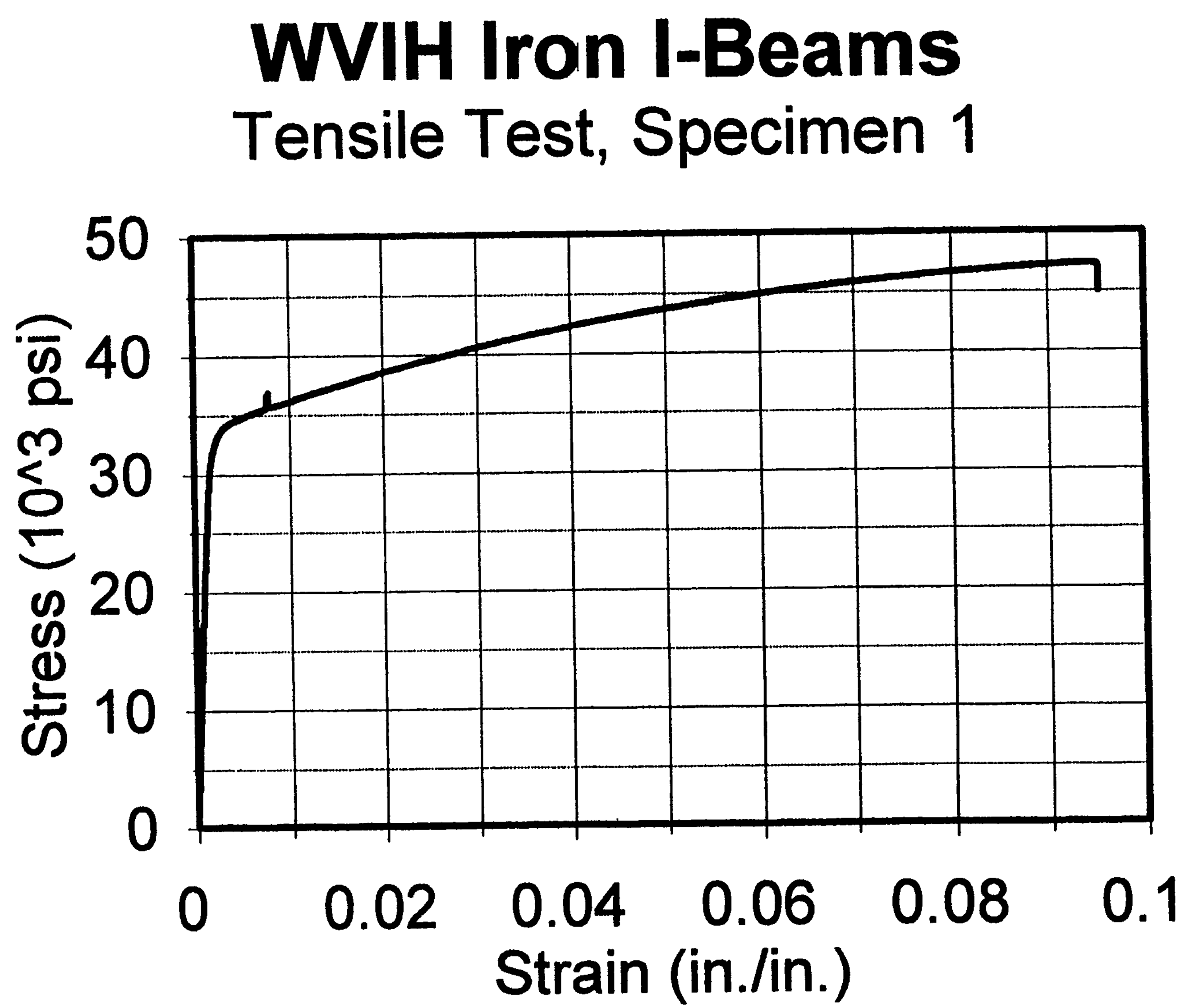
[5]*Specifications for Masonry Structures. "Compressive Strength of Clay Masonry." ACI 530.1/ASCE 6-92/TMS 602-92.*

confirm these minimum strength assumptions. Since all the loads on the system will be factored in accordance with reliability theory, this minimum value will be reduced by a ϕ factor of 0.67. This produces an overall factor of safety of 2 which is reasonable since cosmetic cracking of the brickwork itself is acceptable. Indeed for the system to work as has been proposed it is likely that the system will crack. Natural cement mortar is well known for a high tensile capacity therefore it will be allowed an elongation of 7.5 times the square root of the prism strength, similar to that of concrete.

Cooper & Hewitt 9" Wrought Iron I-Beams

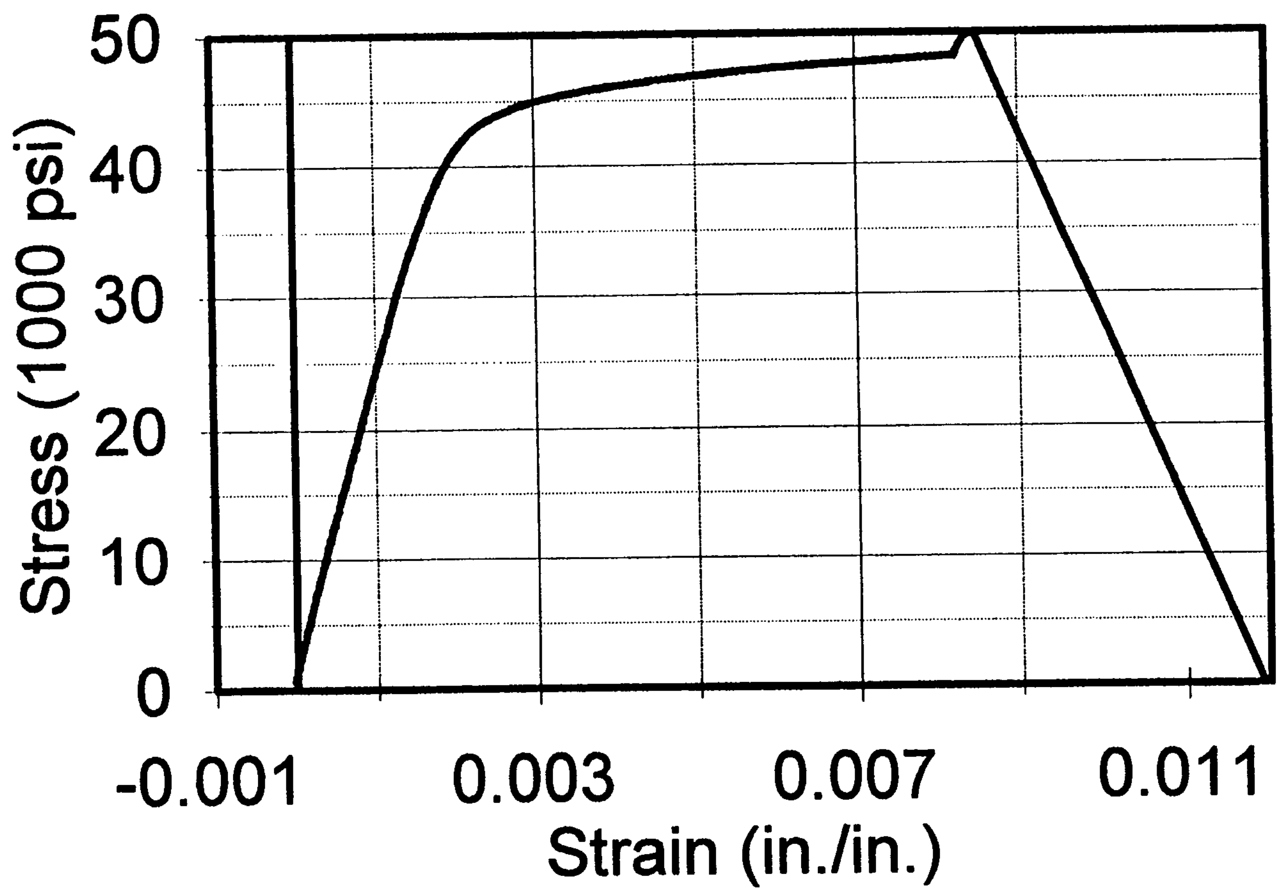
Destructive Testing. Several standard destructive tests have been performed on iron samples from the Wheeling Custom House. Testing included standard structural tensile coupons taken from the web, and 1" diameter compressive samples taken from the transition area between the web and the flange.

Three standard ASTM structural coupons were taken from the web of the I-section to be tested in tension at Turner-Fairbanks Highway Research Station. Sections were 18" long with a 1.5" wide gauge area. Strain was measured with a standard clip gauge providing strain measurement well beyond yielding of the specimen. Sample 1 showed clear yielding and plastic flow though it failed in the fillet zone of the specimen. Sample 2 had a large slag inclusion near the fillet zone and failed in a brittle manner at the inclusion. Even though the failure was apparently brittle both the stress vs. strain curve and the specimen itself showed significant yielding. Sample 3 failed in a ductile manner within the gauge area. All three specimens showed some evidence of both plastic flow and brittle failure. Stiffness was determined through linear regression performed on initial portions of the stress vs. strain curve. Table 5.1 summarizes the engineering values that can be derived from the curves:


WVIH Iron I-Beams
Tensile Test, Specimen 1
Stress (10^3 psi)
50
40
30
20
10
0
0
0.02
0.04
0.06
0.08
0.1
Strain (in./in.)

WVIH Iron I-beams

Tensile Test, Specimen 2



WVIH Iron I-beams

Tensile Test, Specimen 3

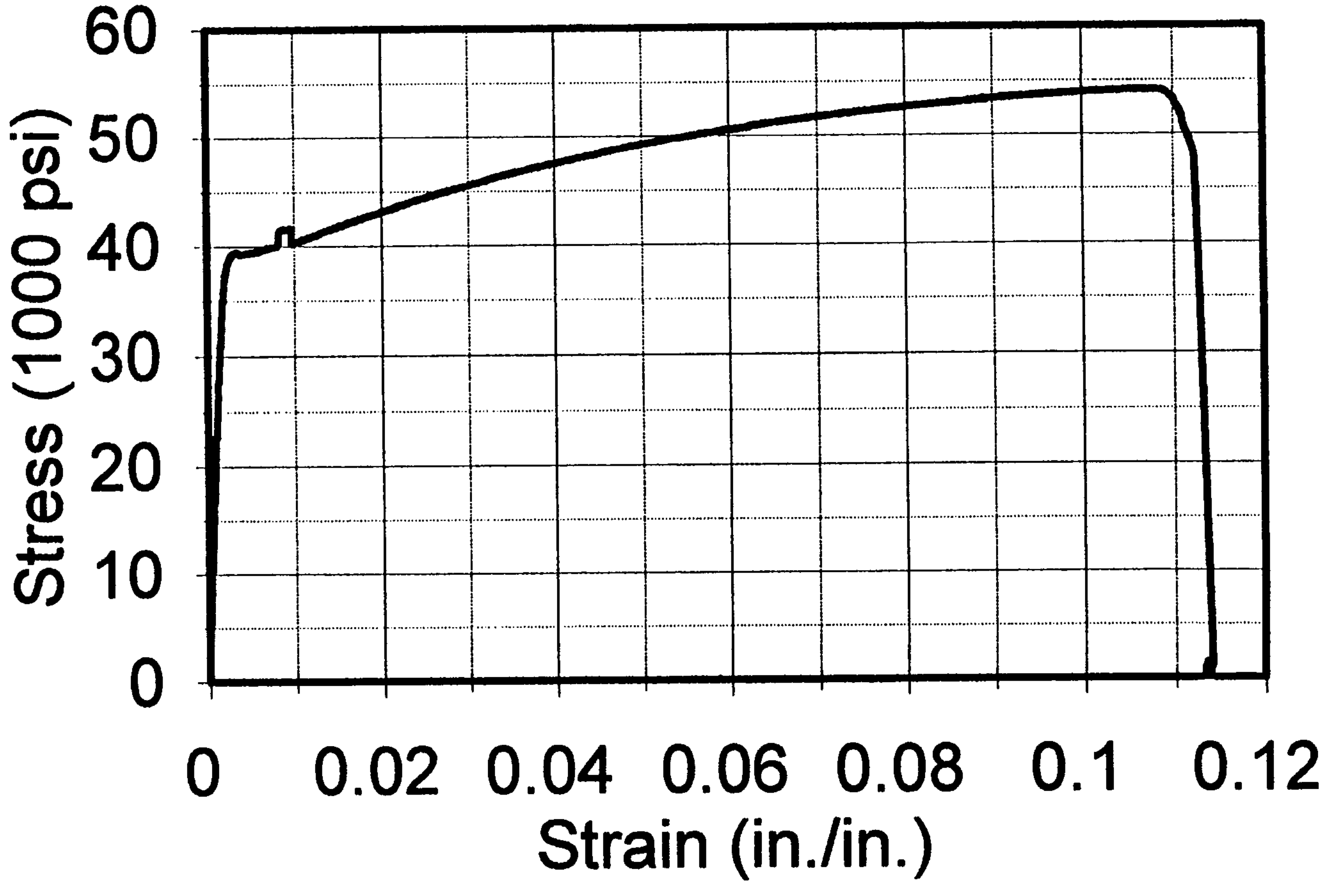


Table 5.1 Results from Destructive Testing of 1856 Iron Specimens

	Young's Mod. (ksi)	Yield Stress	Ultimate Stress	Ultimate Strain
Specimen 1	23,025	34 ksi	48 ksi	0.095
Specimen 2	23,803	42 ksi	48 ksi	0.0087
Specimen 3	25,021	39 ksi	54 ksi	0.11
Average	23,950 (23,743)*	38.3 ksi	50 ksi	0.0712
S.D. (σ)	1,006 (919)	4.04 ksi	3.46 ksi	
COV (σ/m)	0.042 (0.039)	0.105	0.069	

*The values in parentheses include the results from the static compression test

A static loading compression test was performed to verify the results of an ultrasonic, p-wave test. The sample was taken from the transition area between the flange and the web and loaded in the direction of the grain. The test showed a Young's Modulus of 23,124 ksi.

Non-destructive testing. Ultrasonic testing was done on the iron beams both in the lab and at the site. Testing included both p-wave (body wave) and Raleigh (surface) wave sensors. The velocity of the wave depends on the density of the material, Poisson's ratio, and Young's modulus. Having measured the material density and assumed Poisson's ratio, Young's modulus (E) can be determined from measuring the travel time and distance between the sensors. Because wrought iron has a grain structure with a definite grain direction caused by rolling, the stiffness properties may vary in different directions, similar to how timber properties vary in longitudinal and radial directions. The difference is not as pronounced as in timber, but may account for some of the variability in the measurements. Wrought iron is a composite material consisting of ferrite grains and slag. Since these two materials have different stiffnesses and are both elongated in the rolling direction, frequently two seperate peaks in travel time are seen in these tests, especially when taking measurements in the direction of the grain, though it may be hard to confirm these as seperate without a single pulse generator . Since the second peak cannot be relied upon, the first peak is used to determine the stiffness, although this may just be the stiffness of a single constituent in the wrought iron. Obviously the stiffness of the overall specimen is dependent on both constituents. Despite these difficulties, the error in these tests remain within 10-15% for an average of several tests. To clarify the expected error a parameter study follows:

The p-wave (compression wave) test uses sensors that send a signal through the sample. An oscilliscope then measures the time of flight from one sensor to the other. For a p-wave[6]:

$$v_p = \sqrt{\frac{E(1-\upsilon)}{\rho(1+\upsilon)(1-2\upsilon)}}$$

Therefore, rearranging gives:

$$E = \frac{v_p^2 \rho (1+\upsilon)(1-2\upsilon)}{(1-\upsilon)}$$

The variables for this equation are listed in Table 5.2 along with their reasonable error for a length of about 1 inch between the sensors.

[6]Krautkramer, J. and Krautkramer, H. *Ultrasonic Testing of Materials.* Springer-Verlag, Heidleberg. 1969.

Table 5.2 Varibles in P-wave measurement:

Variable	Error	(%)*
Distance:	0.001 m	4
Time:	0.5 μs	12
Velocity:	1100m/s	16
Density, ρ	10 kg/m^3	2
Poisson's Ratio, υ	0.01	3
Young's Modulus, E	12,500 ksi	44

***Possible error percent for a typical 1" measurement**

The major source of error in this case is the distance. As the distance grows, both distance and travel time increase, and the errors become less of a factor in the measurements. The test this parametric study is connected to was performed over a length of approximately 1 inch (2.5 cm) allowing for a large error. Both errors for travel time and distance are only one order of magnitude less than the measured value. As the measured distance and therefore travel time become larger then the velocity can be measured much more accurately. However, increased distance also translates into increased noise and signal attenuation. The travel distance must be at least 3 to 4 inches for the measurement errors to reduce.

Results from several different tests are listed in Table 5.3:

Table 5.3: P-wave test results for wrought iron I-beams.

Distance (m)	Time (μs)	Velocity (m/s)	E (ksi)
0.02447	4.08	5998	28,117
0.02491	4.12	6046	28,574
0.02473	4.10	6031	28,438
0.02430	4.00	6075	28,848

0.2300	39.00	5897	27,186
0.1300	21.8	5963	27,754
	Avg:	6002	28,153
	σ:	58.7	553.3
	COV:	0.01	0.02

The Raleigh (surface) wave test is similar except that it uses a knife edge sensor so that the surface distance can be accurately computed. For a Raleigh wave[7]:

$$v_r = 0.575\sqrt{\frac{E}{\rho}}$$

Therefore:

$$E = \frac{v_r^2 \rho}{0.3306}$$

The variables for this equation are listed in Table 5.4 along with their probable errors:

[7]Krautkramer, J. and Krautkramer, H. *Ultrasonic Testing of Materials*. Springer-Verlag, Heidleberg. 1969.

Table 5.4 Variables and errors for Raleigh wave test

Variable	**Error**	**%**
Distance	0.001 m	1.1
Travel Time	0.5 μs	1.5
Velocity	65 m/s	2.5
Density, ρ	10 kg/m^3	2
Young's Modulus, E	1000 ksi	5

The major difference in the expected errors for this test is that the typical gauge length used is 9 cm instead of the thickness of the sample. As this table shows, the errors reduce significantly although the gauge length is only 4 times larger than that of the p-wave test seen earlier. This is an acceptable error range. Unfortunately the results compare no better than the previous test to actual destructive test results. Again, it is likely that the error is due to the composite nature of the material. Table 5.5 shows the test results from a series of tests done both in the lab and at the site:

Table 5.5. Raleigh Wave tests on wrought iron I-beam:

Distance (m)	Time (μs)	Velocity (m/s)	E (ksi)
0.0762	25.1	3035	29,310
0.0767	25.0	3070	29,996
0.0983	32.4	3033	29,283
0.0900	33.0	2727	23,637*
0.0900	30.0	3000	28,601*
0.0900	31.0	2903	26,782*
0.0900	35.0	2571	21,006*
0.0900	35.4	2542	20,535*

0.0900	35.2	2557	20,775*
0.0900	37.0	2432	18,796*
0.0900	35.4	2542	20,535*
0.0900	34.9	2579	21,137*
0.0900	35.5	2535	20,422*
0.0900	36.2	2486	19,640*
0.0900	35.4	2542	20,535*
0.0900	35.3	2549	20,648*
0.0900	33.5	2687	22,945*
0.0900	36.8	2446	19,013*
0.0900	33.2	2711	23,356*
0.0900	36.1	2493	19,751*
0.0900	33.0	2727	23,638*
0.0900	29.4	3061	29,804
0.0900	29.6	3040	29,403
	Avg:	2707	23,216
	σ / COV		3,939 / 0.17

*Indicates measurements taken in the field.

Although the average of the test results is quite accurate, compared to the destructive test, the standard deviation is very high. Without a large sample size, validity of the results is obviously in question. In addition, if the results are seperated by where they were taken, in the field or in the lab, an interesting difference surfaces. Table 5.6 summarizes the differences:

Table 5.6 Lab Results vs. Field Results For Raleigh Wave Stiffness Determination (ksi)

	Lab testing	Field testing
Average:	29,559	21,766
σ:	288	2,553
COV:	0.01	0.12

Although there is significant difference between the two averages, the COV's are now reduced to reasonably precise levels. Even more interesting is that surface roughness plays little if any role in the measurements. Samples were tested with only the sensor application points treated and then tested with the entire travel length treated and there was no difference between their respective travel times.

However, in the returning wave there were two major peaks observed. The first peak was taken as the travel time, however because of the composite nature of the material, it is possible that one peak corresponds to the travel time through the iron, while the other peak corresponds to the travel time through the slag. This has not been confirmed, but it would explain some of the variation in the measurements from one observer to another.

Metallographic testing. In 1978, researchers at USX performed a chemical and micrographic analysis of the custom house iron. They compared the chemical composition of the sample to a typical hand-puddled wrought iron of the 1850's, according to the 1948 ASM Metals Handbook. The results are listed in Table 5.7:

Table 5.7: Composition of Wrought-Iron-Beam Sample From
U. S. Custom House, Wheeling, West Virginia, percent

	C	Mn	P	S	Si	Slag
Beam Sample	<0.005	0.029	0.610	0.039	0.590	7.4*
Typical hand-puddled wrought iron**	0.06	0.045	0.068	0.009	0.101	1.97

*Weight Percentage based on a volume percentage of 12.5 measured with the quantitative television microscope and assumed specific gravities of 4.5 for slag and 7.6 for wrought iron.

**For wrought iron made before 1930. Reference: Metals Handbook, 1948 Edition, page 504, American Society for Metals, Cleveland, Ohio.

The researchers noted that the high phosphorus and slag contents would be detrimental to the ductility and toughness of the material. They also noted that the ferrite grains were particularly large for this type of iron. The elevated levels of sulfur, coupled with the lower level of Manganese may indicate that the iron was "hot short"--a common problem with early iron that seriously impedes ductility and fatigue resistance[8]. The low carbon content may indicate that the iron grains carry almost exclusively α-iron.

In addition to the analysis done by USX, Dr. Wayne Elban of Loyola University has performed extensive Vickers and Rockwell hardness testing, and prepared numerous micrographs of the iron in several areas of the cross section[9]. He also recognized the large grain structure and large slag inclusions located typically around the grain boundaries instead of inside the iron grains themselves. This would tend to indicate that there was no heat treatment after the rolling process.

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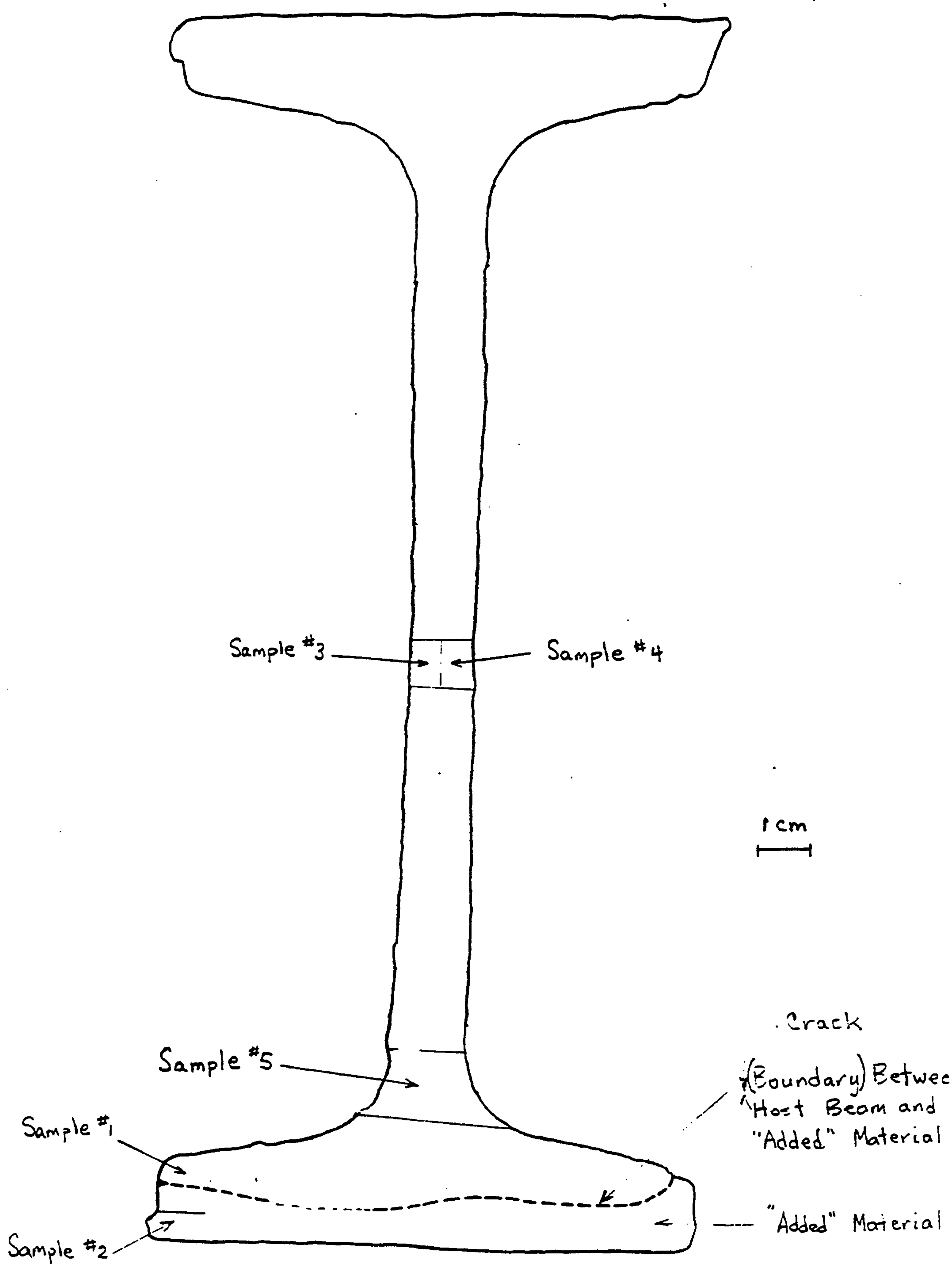
[8] *Aston, James and Story, Edward B. Wrought Iron. A.M. Byers Company, Pittsburg, Pa. 1957. p 45.*

[9] *Elban, Wayne. Report to IHTIA on metallographic testing. Currently being submitted for publication.*

Both grains and slag particles are elongated along the rolling direction as would be expected. He was also able to distinguish a thin, uniform layer of slag at the grain boundaries themselves. This layer of slag is visible in polarized light which indicates either a different chemical composition or a cubic crystal material (slag) under strain. The inhomogeneous nature of the microstructure is also evident in the hardness values. The extreme variation in the hardness values were probably caused by local slag inclusions and localized work hardening. The slag inclusions are quite brittle, and radial cracks were seen in almost all Vicker's impressions indicating some brittleness in the iron matrix itself. Cracks were also seen a large slag particle that were not attributable to the impressions.

Of greater significance to the overall stability of the system, it was discovered that there is a substantial difference between a piece of material on the outside of the flange than on the inside. The boundary between the two materials is clear on the surface of the cross section, and is partly filled with slag. The boundary does not seem to extend through the entire width of the flange, therefore the section will still be treated as homogeneous. The failure mode for that piece becoming disconnected would likely be in shear near the supports, although shear levels should be relatively low since the boundary is approximately 1/4 inch away from the surface of the flange. Since the moment capacity required at the supports is low, the reduction in effective cross sectional area is negligible. Even in the worst samples there is at least 3 cm of apparent welding between the internal and external flange. There is a substantial difference in hardness between the interior and exterior flange which seems to indicate that the exterior flange may have been added late in the manufacturing process. The layer of billets used in the rolling process had several interior layers that were the same width along with top and bottom layers that were slightly wider. This seems to indicate the probability that the interior billets were run through the mill first until a flange was formed approximately the same width


Sample #3
Sample #4
1 cm
Crack
Sample #5
(Boundary) Betwee
Host Beam and
"Added" Material
Sample #1
Sample #2
"Added" Material

as the top and bottom layers. Then these layers were added and the forming process continued. It is known that Trenton Iron was having substantial problems forming the I section and the flanges frequently sheared off in the rolling process. This may have been the only way to correct that problem.

Conclusions

Material properties for the brick will be assumed as:

f'm	1800 psi
ν	0.11
max elongation:	0.0003
Modulus of Elasticity	1,800,000 psi
Modulus of Rigidity	720,000 psi
f't	320 psi

Material properties for the iron will be:

Young's Modulus:	24,000 ksi
Yield Stress (f_y):	38,000 psi
Ultimate Stress (f_u):	50,000 psi
Design Shear Strength (V_n)[10] full x-section:	90,000 lbs
Design Shear Strength (V_n), cut section:	34,500 lbs

[10]*Taken from design provisions for Steel, $V_n=0.6F_{yw}A_w$. AISC, LRFD Specification. 1986. Section F2.*

Chapter 6: Safety

Introduction

One objective of this project is to establish the safe loading capacity of the structure. Frequently, preservation architects and engineers assume that since the building has performed satisfactorily in the past, it will continue to do so in the future. In fact, this may not be true for several reasons. Degradation may significantly affects the capacity of the system, but it is not the only factor that must be addressed. Other factors like material behavior and variability must also be considered. Historic materials do not behave like modern steel or concrete, and in comparison to the volumes of information available on these modern materials, there is little data to suggest how their response could differ. These differences could substantially affect the amount of safe capacity available in the system. In addition, historic systems, like the one in the Wheeling Custom House, may be more or less redundant than modern structural systems. Even failure modes may need reconsideration when the system is drastically different than modern building systems.

Background

One of the issues to address is the lack of information surrounding historic materials. In a modern analysis, the typical engineer depends on the established codes of practice and material specifications to provide him with an acceptably conservative design and analysis methodology based on the statistical properties of both the material and the loads. With a historic material, this type of information is not readily available. Even when a design aid similar to the 1889 Trenton

Iron handbook[1] is available, the material properties listed may or may not have statistical validity. The properties listed may have been adjusted by the manufacturer either to provide a safer design or to appear more competitive. Occasionally test results will be available that list both load and deflection at intermediate loadings so that the material stiffness can be derived, but frequently only a breaking load is listed while intermediate deflections are ignored. Even where deflection results are given, they are typically recorded to the nearest 1/32 of an inch. Although this is a small measurement, it is at least 1/3 less accurate than results obtained through current testing procedures. At Trenton, tests were frequently repeated throughout the manufacturing as a quality control measure[2], but those test results are not tabulated or statistically analyzed to our knowledge. When dealing with other samples from that era, it must be recognized that the practice of routine testing was far ahead of its time. Other manufacturers that are contemporary with Trenton may not have the same level of material consistency assured by constant quality checks.

In the absence of significant corrosion, it is likely that the material properties for the iron will stay consistent over time. However, this is clearly not the case for brick and mortar. Although well-made masonry can retain substantial strength for several centuries, these materials are adversely affected by time. More in-situ research is required to understand these degradation

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[1] *Useful Information for Engineers, Architects, and Constructors..."* New Jersey Steel and Iron Co.; Cooper Hewitt & Co., St. Paul, Minn. 1889.

[2] ibid. pp. 66m-66o. By 1889, 4 destructive tests per shipment were required by the "leading bridge builders" with an additional test for every 50,000 lbs of steel shipped. Samples were rejected when failing to meet the minimum requirements by more than 4%, and a 20% rejection rate was sufficient to reject the whole lot.

Nevins, Allan. *Abram Hewitt*, Harper & Brothers, New York, 1935. p. 90. Consistant testing was started at Cooper & Hewitt very soon after its inception. Nevins writes: "A special employee was retained in 1848 to have charge of the puddlers on each turn, watch every heat squeezed, and report 'what the quality, the number of bad balls, and the reason why the iron was bad.'"

effects, as well as how water and other environmental factors affect their aging. Experience demonstrates that well-burnt interior masonry can retain most of its strength for upwards of 500 years, but it might be unwise to assume this is the case when dealing with site made or ill-burnt brick, especially in poor environmental conditions. Destructive and nondestructive testing can be used effectively to help determine the current state of strength, stiffness, and stress in a component. Testing several areas of an existing structure can provide the statistical properties necessary to estimate the reliability of the system. It is preferable to perform in-situ testing if possible to get prism properties that reflect the interaction of both brick and mortar. Substantial research into this topic has gone on in Europe, perfecting several techniques that can provide reliable results.[3] In lieu of prism testing like the flat jack test, tests on individual bricks can provide an estimate of the strength of the system. Wherever economical, these tests can provide the type of statistical information necessary to assure the engineer of the reliability of the system.

Discussion

To assure the safety of the system target reliability indices will be selected for the major failure modes involved in this system. Appendix A has a summary of reliability theory and Appendix B gives a discussion of the factors that play a part in determining the reliability index that is required for a system.

Failure due to yielding would cause serious destruction to the structure, and would require intermediate supports to correct. This could make the Custom House unusable even as a museum, depending on where the failure occurred. Because the fracture load could be very early

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[3]Rossi, Pier Paolo, "Non-destructive Evaluation of the Mechanical Characteristics of Masonry Structures," *Proceedings of Conference on Nondestructive Evaluation of Civil Structures and Materials,* University of Colorado, Boulder, CO, October 15-17, 1990. pp. 17-41.

in the plastic range, loading into the inelastic range should be avoided because of the potential of fracture. Since all the jack arches depend on each other for lateral support, the loss of one iron beam could cause a cascade failure of that floor system. The dynamic effects of the collapse could also precipitate the failure of the columns and lower floor systems, therefore causing the collapse of the entire structure. Fortunately, previous experience with this type of system indicates that it takes several hours of excessive loading for yielding to occur and there are obvious signs that accompany the failure like excessive deflection. This should provide sufficient time to reduce the load and correct the problem. Based on the consequences of failure, a β value of 3.5 will be chosen against yielding, and a β of 4.0 against shear failure and fracture of the iron.

Since we have assumed the system behaves in a partially composite manner, another possible limit state is the crushing failure of the brick. Crushing due to the thrust force in the direction of the arch could cause instability in the system and precipitate collapse, however, some warning of overload of the arch could be apparent. A β value of 3.5 is indicated, since failure could cause complete destruction, but the potential for loss of life is uncertain. Crushing due to the bending of the composite system is a less serious failure mode since there is ample area of iron in the top flange for redistribution of force. Therefore a β value of 3.3 is indicated, in part because of the lack of information available. Instability in the arch is another failure mode to be considered. Since instability in one barrel vault would reduce or eliminate the thrust force supporting the adjacent vaults, the failure mode is similar to crushing in thrust and is therefore given the same reliability index of 3.5.

Since a Level II analysis is required, typically a First Order, Second Moment analysis would be performed to check the actual reliability of the system with the minimum desired. The formula to determine β for normal and lognormal distributions are as follows:

$$\beta_N = \frac{\bar{R}-\bar{Q}}{(\sigma_R^2+\sigma_Q^2)^{1/2}}$$ Normal Distribution

$$\beta_{LN} = \frac{\ln(\bar{R}/\bar{Q})}{(V_R^2+V_Q^2)^{1/2}}$$ Lognormal Distribution

Where:

$\bar{R}$ is the mean of the resistance

$\bar{Q}$ is the mean of the load effect

σ^2 is the standard deviation

V is the coefficient of variation

Although the method is not very complicated, the statistical information concerning the load distributions is not widely known, and the methodology involved in Level II analysis is not familiar to most practicing engineers. It is therefore desirable to provide the designer with the type of factors used in a Level I design methodology if possible. Resistance factors can be determined according to the approximate formula[4]:

$$\phi \approx \exp(-\alpha\,\beta\,V_r)$$

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[4]Putcha, Chandra; GangaRao, Hota; et.al. *Reliability Analysis of Stressed Timber Bridges.* NSF Project Report, Washington, DC, 1993. p. 20.---May need to list others.

Where:

α is a direction cosine to the failure surface (1 is conservative)

β is the desired minimum reliability index

V_r is the coefficient of variation for the material resistance.

Conclusion

Table 6.4, below, summarizes the failure modes and the factors to be used:

Failure Mode	β	COV	ϕ	$\bar{R}$	$\phi\bar{R}$
Iron Bending Stress, Yield	3.5	0.105	0.69	38.3 ksi	26.4 ksi
Iron Bending Stress, Fracture	4	0.069	0.76	50.0 ksi	38.0 ksi
Iron, Shear, full cross section	4	0.105*	0.66	90.0 kip	59.4 kip
Iron, Shear, Cut beams	4	0.105*	0.66	34.5 kip	22.8 kip
Brick, Crushing due to Thrust, Arch instability	3.5	0.248	0.42	1800 psi	756 psi
Brick, Crushing due to bending	3.3	0.248	0.44	1800 psi	792 psi

*Shear COV's are assumed the same as yield due to lack of information. The value is reasonably conservative.

The load factors are also dependent on the reliability index and can be derived in a similar manner. However, as mentioned earlier, the coefficient of variation for typical building loads is not widely known and the nominal loads listed in ASCE 7-88 are not the true mean, and therefore must be adjusted if load factors are to be derived. As stated earlier, current practice uses the loads and the inherent reliability given by ASCE 7-88 despite the slant away from the target reliability. These factors are:

1.2 for dead load

1.6 for live load

Environmental loads have not been considered in this design, therefore importance factors are not applied. There is some concern about the system's behavior in a seismic event due to the amount of masonry work in the structure, both in the jack arches and the exterior loadbearing walls. The seismic loads that are applied by the code for this region are very low, and are typically considered negligible for this type of historic structure due to the probability of the structure having already survived any reasonable seismic event in its lifespan. Since there is no record of any structural damage to this building due to a seismic event, and the dynamic characteristics of the system are not likely to have changed in its history, it is reasonable to conclude that it should be able to withstand any reasonable future earthquake. However, a thorough analysis should eventually include seismic effects as a load case.

Wind ratings on this location are 70 mph, the lowest rated sustained wind speed, and are also negligible. The roof has been renovated within the last 30 years and therefore was required to meet code requirements for snow loads.

Chapter 7: Description of the Structural System

This chapter deals with the structural system used in the Wheeling Custom House. First, the structural system is described in detail, following the load path from the roof to the foundation, and the crucial structural members are highlighted. Next, the previous analysis of the floor system capacity is detailed to show how the analysis may be incomplete. A description of the system's behavior and the analysis procedures to be used in this study is also provided.

Load Path

The roof is supported by a truss composed of eyebars and rail iron possibly imported from Wales. Although the system is not completely original, it does date from after the Civil War. Since there is no obvious distress in the roof system, and there is no reason to expect higher roof loads than was experienced in the last hundred years, it is assumed to be adequate. In addition, the roof has been remodeled and brought up to code standards within the last 30 years as part of the building's ongoing restoration.

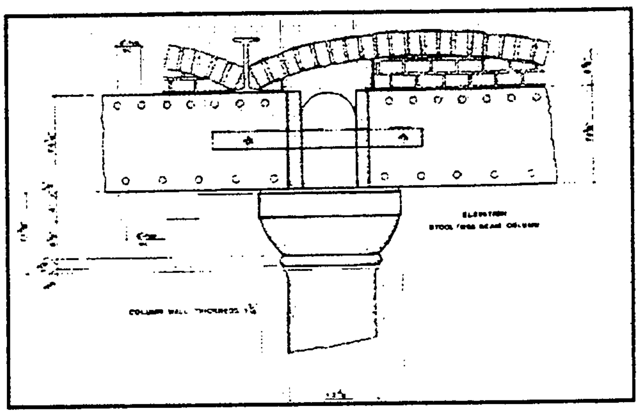

Figure 7.1

The exterior loadbearing walls are common stone masonry walls, well within the 1 in 12 unbraced thickness guideline for unreinforced construction. The interior floors consist of wrought iron I-beams spaced at 51 to 53 inches on center. Low segmental arches (jack arches) span between these beams providing support for a wood floor (Figure 7.1). The arches have a rise of about five

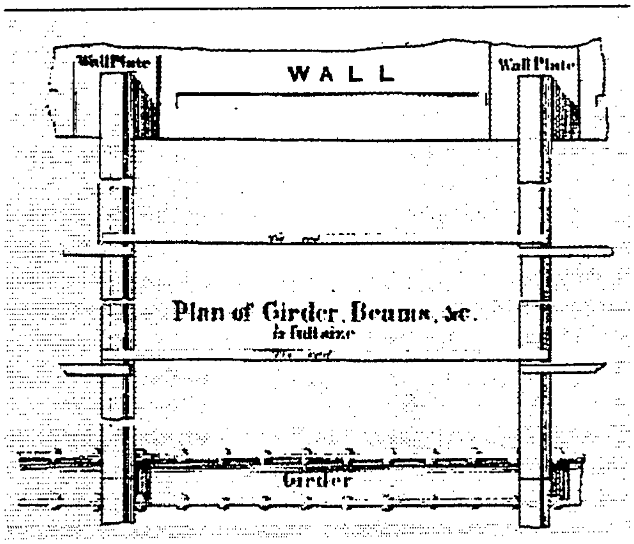

Figure 7.2

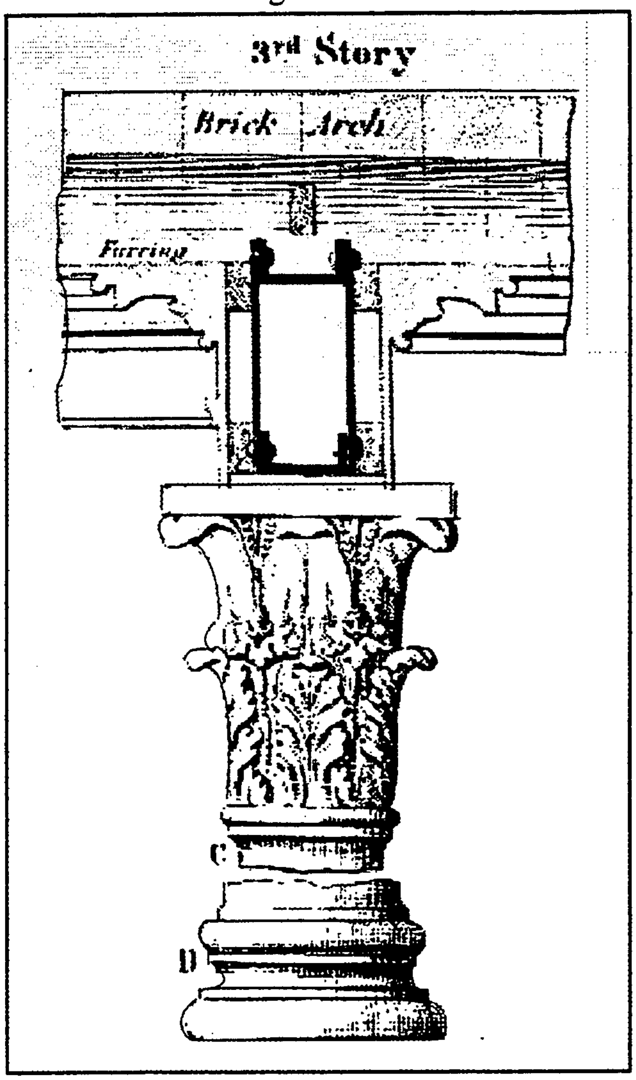

Figure 7.3

and a half (5.5) inches and the iron spans either 15 or 20 feet, depending on its location in the building. Previous measurements indicated that the rise was approximately five inches, but this failed to take into account a 1/2" layer of plaster. The I-beams are supported by riveted box girders in the interior and by the masonry wall along the exterior. The box girders rest on cast iron columns which are of different styles and sizes for each floor. For instance, in the basement the columns are 12 inches in diameter and done in the Doric order. On the first floor, the columns are again round, but of the Corinthian order. The second floor uses 14" square, engaged Corinthian columns that are built into the walls for the side rooms, similar to pilasters. The freestanding columns in the courtroom on the third floor are round, while engaged columns are square and appear as pilasters. The columns are connected from floor to floor and rest on a stone foundation that has been obscured by a concrete floor that dates from the 1960's.

The exterior masonry wall supports the interior skeletal frame. It is an ancient technology that was obviously constructed with at least minimum skill and has shown no structural distress. There have been problems with minor cosmetic spalling, but nothing that could bring its integrity into question. Likewise, although the casting on the columns is obviously poor, they show no major distress that cannot be traced back to original casting problems. These casting defects are only obvious in the basement columns and their vertical load greatly exceeds any moment that may be introduced in them by non-uniform loading of the floor system above, hence there is no tension in them.

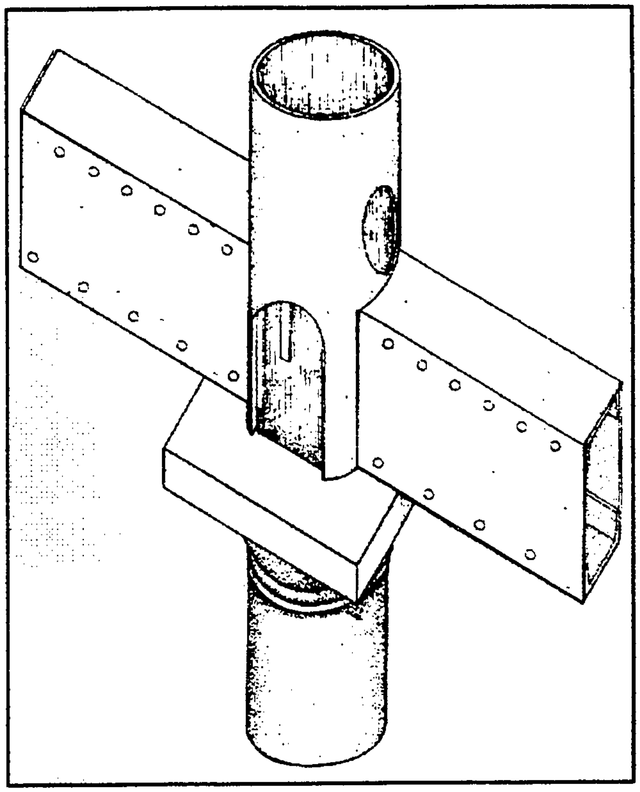

Figure 7.4

System Behavior

The primary interest in this investigation is in the floor system itself. Previously, the brickwork was seen as a load transfer component and therefore not assumed to add any structural capacity to the system. This is clearly not how the system behaves for two reasons. If the iron were the only structural component involved, it is likely that the system would have been overloaded and collapsed at some time either during the civil war when the building was used to store munitions or while the building was owned by an insurance company and very likely (literally) overloaded with paperwork.

Assuming that the iron is the only structural component, a summary of the previous analysis is listed in Table 7.1.[1]

Table 7.1, Summary of Previous Structural Analysis

Load Factors (no resistance factors applied):	1.9 Dead Load, 2.3 Live Load
Dead Load:	400 lb/ft of length
Ultimate Strength (assumed):	35 ksi
Iron I-beams:	
Moment of Inertia (I):	125 in^4
Span, outer rooms:	20 ft
Maximum Shear Force (DL only):	4000 lb
Maximum Shear Stress (DL only):	0.91 ksi
Maximum DL Moment:	20 k-ft
Allowable Live Load:	74.4 psf
Live Load deflection:	0.38 in (L / 630)
Span, Interior rooms:	15 ft
Maximum DL Moment:	11.2 k-ft
Allowable Live Load:	184 psf
Box Girders:	
Area:	18.37 in^2
Moment of Inertia (I):	559 in^4
Shear stress (DL):	0.91 ksi
Allowable Live Load:	114 psf
Columns:	
Applied load:	22.3 kip
Compressive stress:	1.1 ksi
Allowable stress:	80.0 ksi

The load factors correspond to a ϕ factor of 0.68 for a DL/LL ratio of 0.4 using the normal

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[1]Emory Kemp, ed. In house analysis done for the Institute for the History of Technology and Industrial Archaeology.

1.2 DL factor and 1.6 LL factor. This is quite comparable to the ϕ factors that will be used later on in this analysis. In addition, the 35 ksi yield stress assumed for this previous analysis is less than the 38 ksi average yield stress to be used in the current analysis for the I-beams. We must conclude therefore that this previous analysis is adequately conservative, except that it fails to include the contribution of the arch work to the strength of the system, making that portion of the analysis overly conservative. Therefore the only portion of the analysis that needs major reconsideration is the beam/arch system.

The stiffness of the brick in the system is at least an order of magnitude higher than that of the iron. This means that the brick deflections should control the overall deflections of the system. Therefore it is likely that the brick itself is carrying the bulk of the load sustained by the system. The tests performed at Trenton Iron in the 1850's clearly show that the addition of the brick arches significantly affects the ultimate capacity and stiffness of the system[2]. In effect, the system acts as a shallow composite beam, like reinforced concrete. To be able to assume composite action in this system it is important to validate force transfer longitudinally from the brick to the iron. Since the brick is assumed to carry little tensile force, a method for transferring that tensile force from the brick to the iron must be available. It is clear that for such a low arch system, the horizontal thrust force becomes very high. This thrust force provides a lateral normal force between the brick and the iron that allows us to take advantage of the friction between the iron and the brick to transfer the longitudinal tensile force. Although this thrust force is probably over 90% of the vertical load that is applied, this force may not be enough to assure perfectly composite action. Even partial composite

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[2]Alexander, B.S. "Wrought Iron Beams" Ex. Doc. No 56., House of Representatives, 33rd Congress, 2nd Session.

action, however, would add a significant contribution to the overall strength of the system.

It is also important to realize that the construction of the arches themselves affects the stress distributions in both the iron and the brick. Initially, centering is placed on the bottom flanges of the I-beams and the brick work is then formed from web to web. This causes an initial deflection and stress distribution in the iron that is not experienced by the brick, since the wet mortar has no capacity to carry stress. When the mortar hardens, the assemblage still carries no stress but has increased stiffness. Therefore the brick only sustains the stresses caused by the infill and the live load, while the iron carries the entire dead load of the brick and the iron as well as a portion of the infill and live load. Because this is essentially the same as unproped formwork construction, the composite action is only threatened by the stresses from the infill and the live load, yet still has the horizontal thrust of the brick to provide a normal component needed to take advantage of the friction between the brick and the iron. The coefficient of friction for mortar is approximately 0.6, not taking into account the roughness of the iron surface (which is substantial) and the bond between the mortar and the iron. In actuality, the adhesive bond strength alone more than accounts for the load transfer between the components. The bond strength for smooth bars is listed in early manuals as approximately 20 percent of the compressive strength of the concrete or mortar. Allowable transfer strength was then established as 2/3 of this bond strength[3]. For this system, the allowable transfer strength is around 240 psi. After this is exceeded the frictional forces involved govern the load transfer. This will be checked in the finite element analysis. Typically transfer stresses are only large near the ends. Since the moments are minimal and the thrust is maximum at the supports this is not a major concern.

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[3]Hool, George, Johnson, Nathan, and Hollister, S.C. *Concrete Engineers Handbook*. McGraw Hill, London. 1918. pp 265-268.

Often the bond between concrete and steel is listed in terms of a minimum transfer length. As long as there is sufficient length to develop full composite action the transfer stresses in the remaining portion of the span reduce to negligible levels.

Chapter 8: Structural Analysis

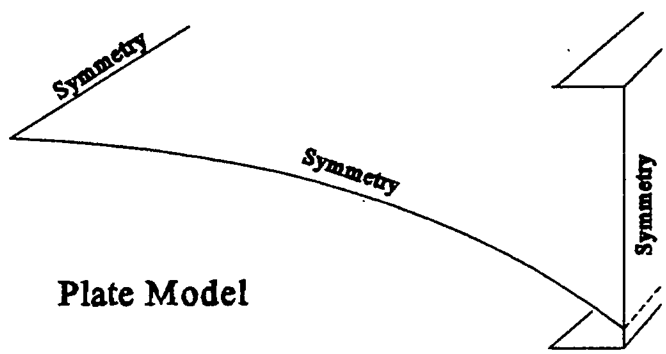
The analysis proceeds in two major phases. A linear finite element analysis is used to establish the stress distribution in the brick and iron and determine whether true composite action is possible for this system. The system is then analyzed as a composite beam using cracked transformed section analysis. All loads are factored according to ANSI/ASCE 7-88 load factors (e.g. 1.2 Dead Load, 1.6 Live Load) and system reliability is confirmed using the resistance factors derived in the previous section. The analysis takes into account both cracking of the brick and the stress distributions due to construction processes.

FINITE ELEMENT ANALYSIS

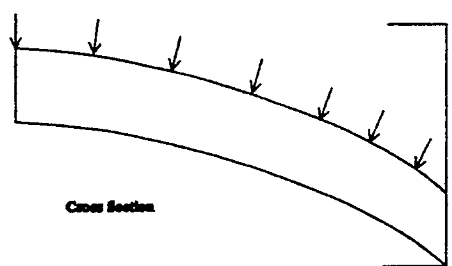
Earlier models

Finite element analysis is a computer modelling technique that involves discretizing a system into smaller "chunks" and analyzing each piece simultaneously to arrive at the system's behavior. The analyst inputs the geometry, material properties, and loads for the system and decides on the size of the divisions that will be made. To model a system properly, the engineer must have a good understanding of the behavior of both the system and the model. Minor details in the model may have a large impact on the way it behaves. Choosing appropriate modelling conditions is an art form that often requires iterative attempts. Each successive model contributes to the understanding of the system and is then further refined to provide as close a picture of the system behavior as is desired. In this analysis there have been at least five major model approaches and innumerable refinements.

The first model idealized the system, both iron and brick, as shell elements. This was only a rough approximation because although the iron is sufficiently thin to use a normal shell approximation, any reasonable mesh requires shell elements for the brick with thicknesses that approximate the width of the elements.



Thus, thin shell elements are invalid. There are formulations for shells that include shear effects, but they are rare and still would not provide a fine enough mesh to provide acceptable answers. Another problem with the model was that the load transfer from the brick to the iron had to be done at a single point along the web of the beam and therefore could not provide good information about the shear stresses at the transition.



The second model used 20 node solid elements to model the brick and plates to model the iron. Loads were transferred to the brick perpendicular to the arch surface and were calculated from an average of the infill depth and a uniformly distributed live load. This was insufficient because the purpose of the model was to determine the horizontal thrust force of the brick. Although the self weight of the brick was applied gravitationally (and therefore correctly) the live load tended to provide resistance to the thrust force.

The third model added orthotropic solid elements to simulate the infill load. The infill was given very little stiffness to avoid affecting the overall stiffness of the system. The infill then loaded the brick system gravitationally and provided a flat horizontal surface on which to apply the live load. The infill was connected only to the bricks and not to the iron as an additional precaution against any additional stiffening due to the infill. This model provided clear load distributions due to bending, but could not take into account the possible cracking behavior of the brick. Unfortunately, although there are element formulations that can model cracked behavior, they are largely first order elements that use linear distributions to determine the shape and stresses in the elements. The computer capacity limitations here prevented using a fine enough mesh to avoid the problems with this type of element. Attempts to model the system with this type of element produced poor results. Other attempts to model the cracked area with a zone of orthotropic material similarly failed. Although the cracked area will affect the behavior of the system, several stress distributions were used in lieu of other sources. The compressive bending stresses in the brick agree well with the cracked transformed section analysis. The shear stresses in the brick in the transition region were also used to establish composite action between the brick and the iron.

Model Details:

The final model was analyzed using Algor's[1]20 node solid. The model used symmetry boundary conditions on three sides to reduce the overall model size. The support for the I-beams was modeled as a vertical translational constraint on the end nodes of the flange of the beam. It is better to spread the support out over several elements, however this was initially difficult because of the mesh size and not necessary since that area is not of great concern. There is a stress concentration that appears there due to this, but the stresses reduce to less than a quarter of the peak stresses within two elements. The real system is similar to a knife-edge support and is therefore somewhat similar to the model in that respect.

A density of 50 pounds per cubic ft was assigned to the infill and factored. A load of 100 psf was also applied to the infill surface. Several plots of the stress distributions follow. At 142 psi, the shear stress on the brick face is well below the 240 psi allowable. The stresses in the longitudinal (x) direction are reasonable for this system and the deflection of 0.56 inches is relatively low for that span, but not unexpected for such a stiff structure. The stresses in the radial direction are also low, with tensile forces well below crack values.

The finite element (FE) model does show some discrepencies from the CTS model, but this is largely due to a more even distribution of the material vertically in the FE model. The CTS model shows much more material in the compression zone than the FE model therefore the neutral axis is shifted much higher. The stresses are higher in the CTS model but substantial increases in stress in the FE model still do not exceed the stress limitations set by this analysis.

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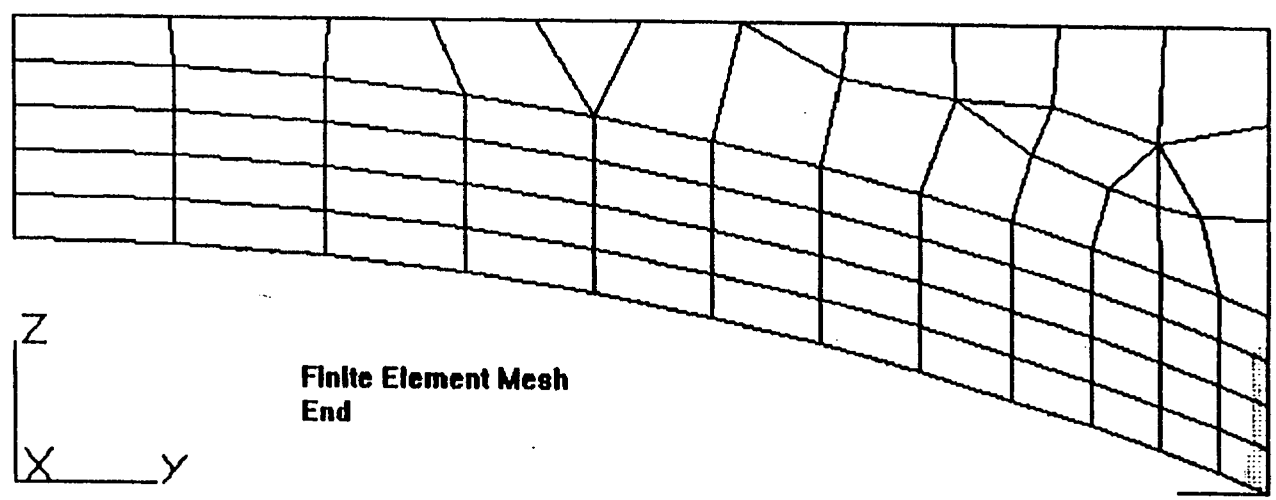
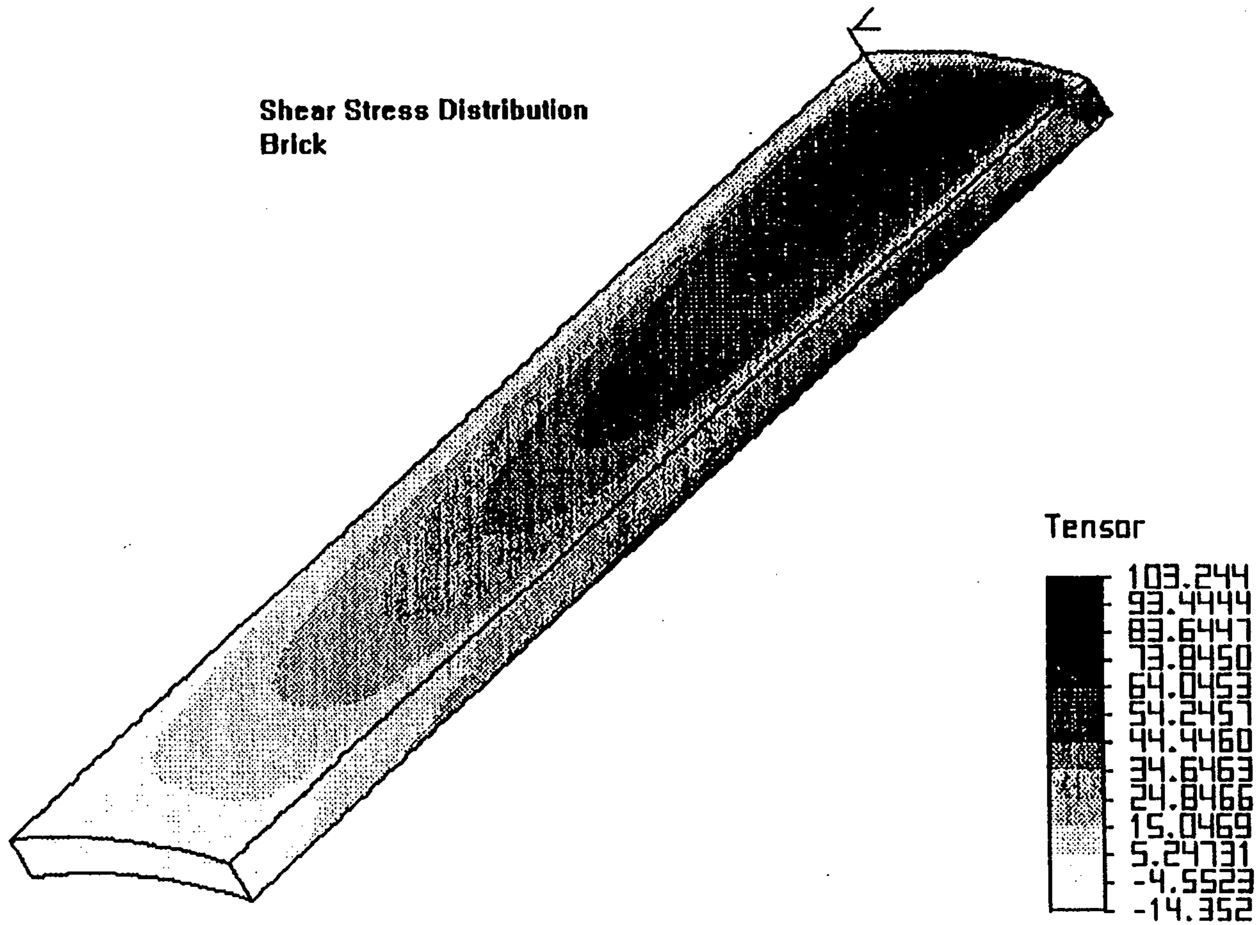
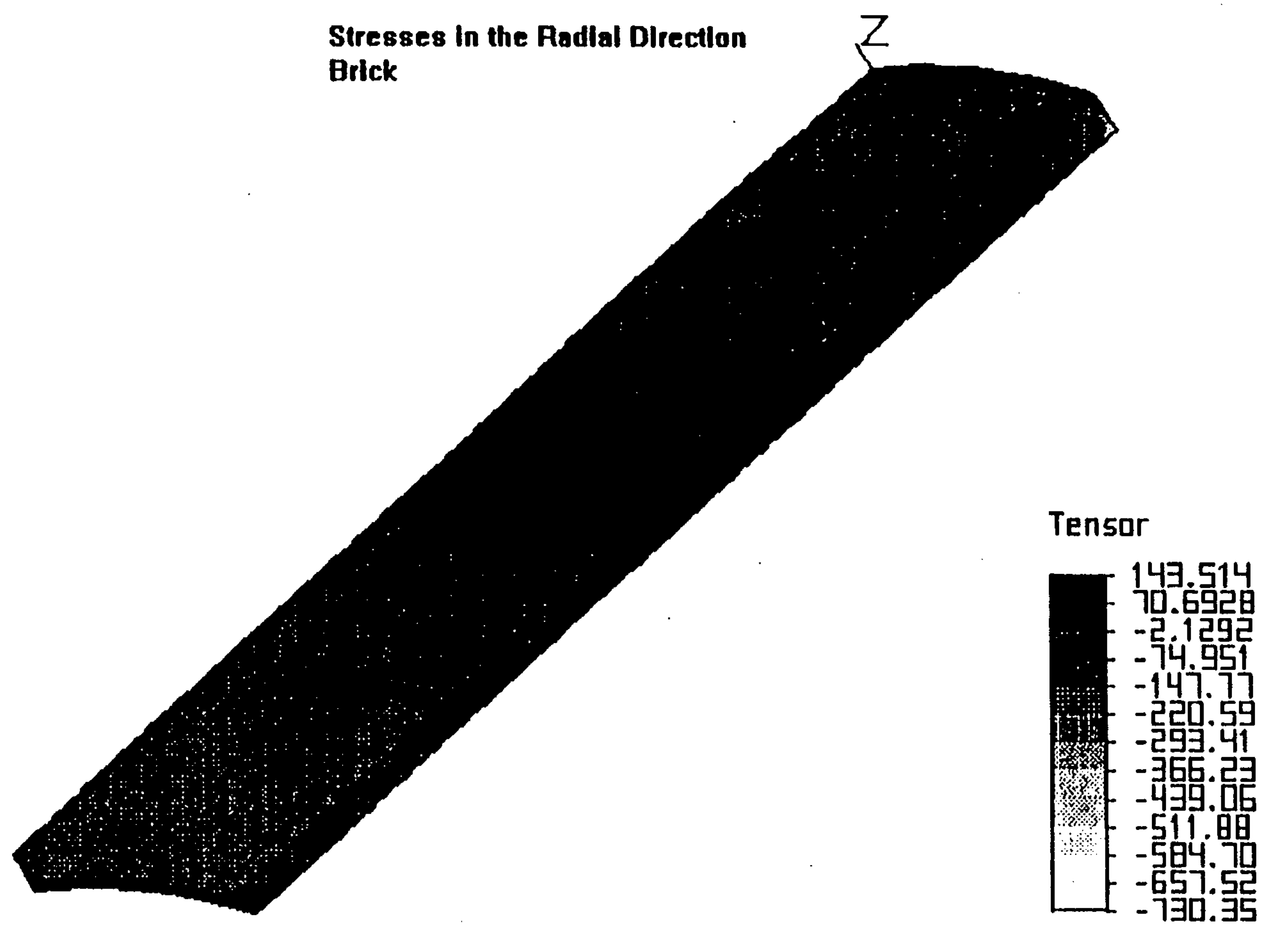
[1]For information on Algor phone (412) 967-2700, or write: Algor Inc. 150 Beta Dr, Pittsburgh, PA 15238-2932.

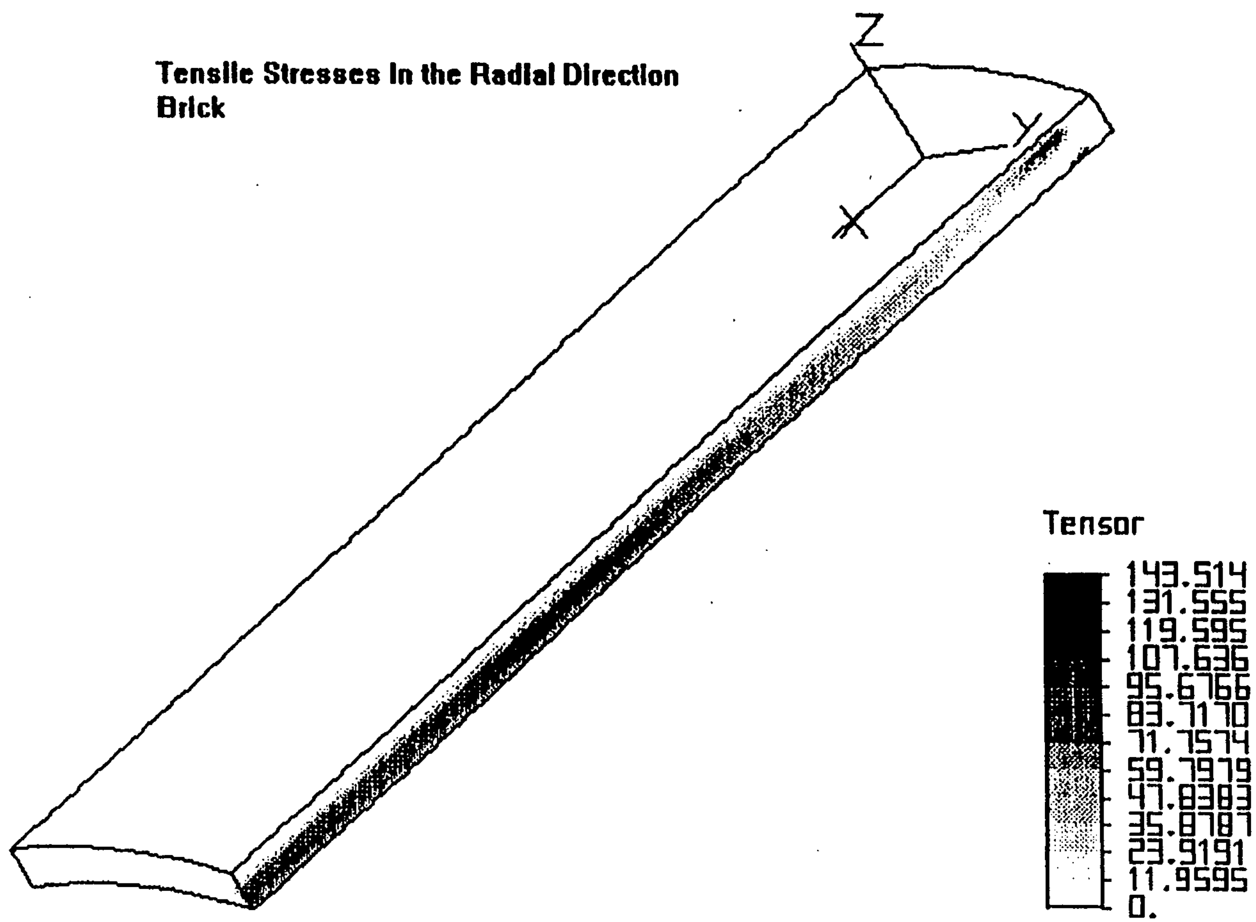

Z
X
Y
Finite Element Mesh
End

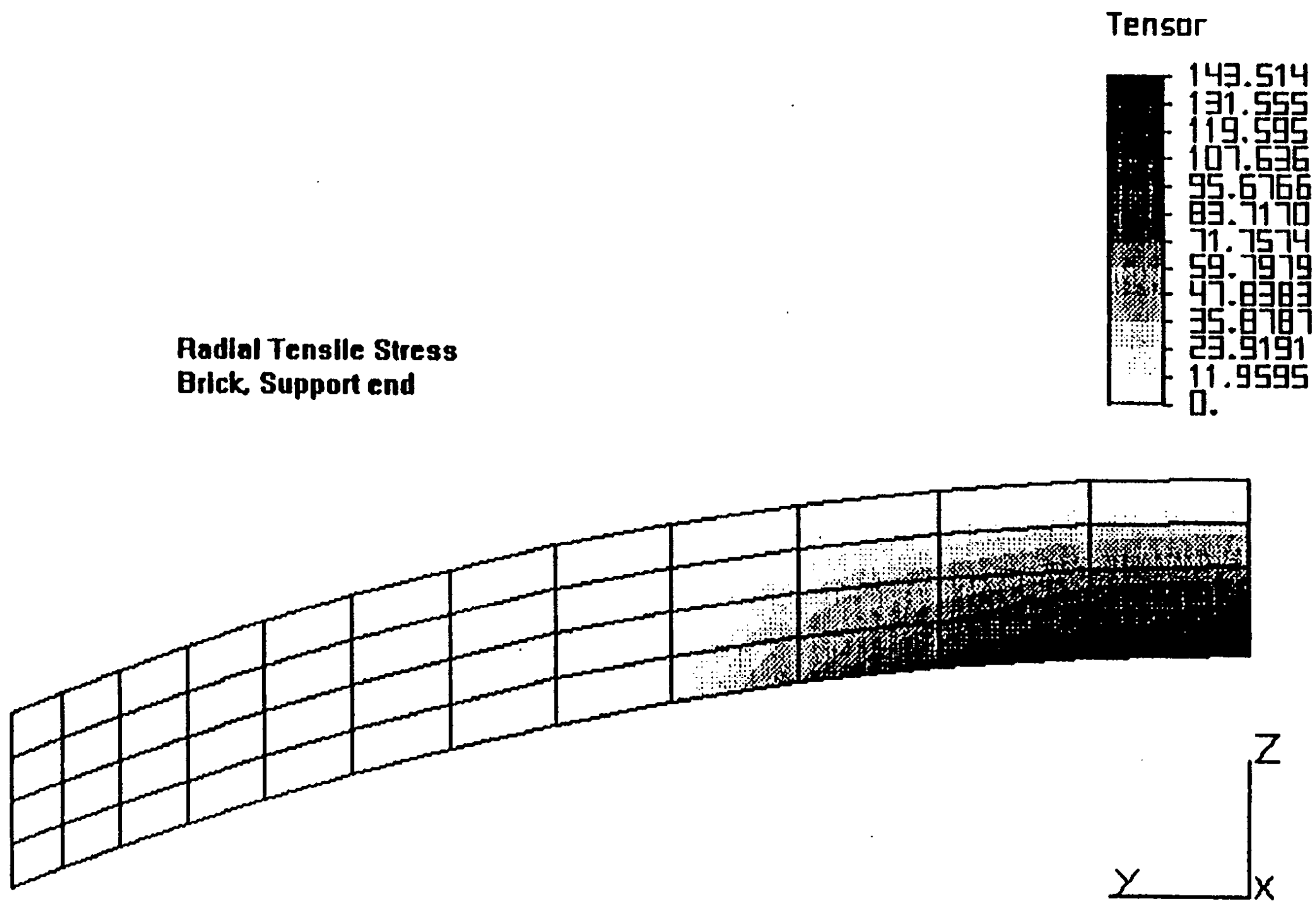

Shear Stress Distribution
Brick
Tensor
103.244
93.4444
83.6447
73.8450
64.0453
54.2457
44.4460
34.6463
24.8466
15.0469
5.24731
-4.5523
-14.352

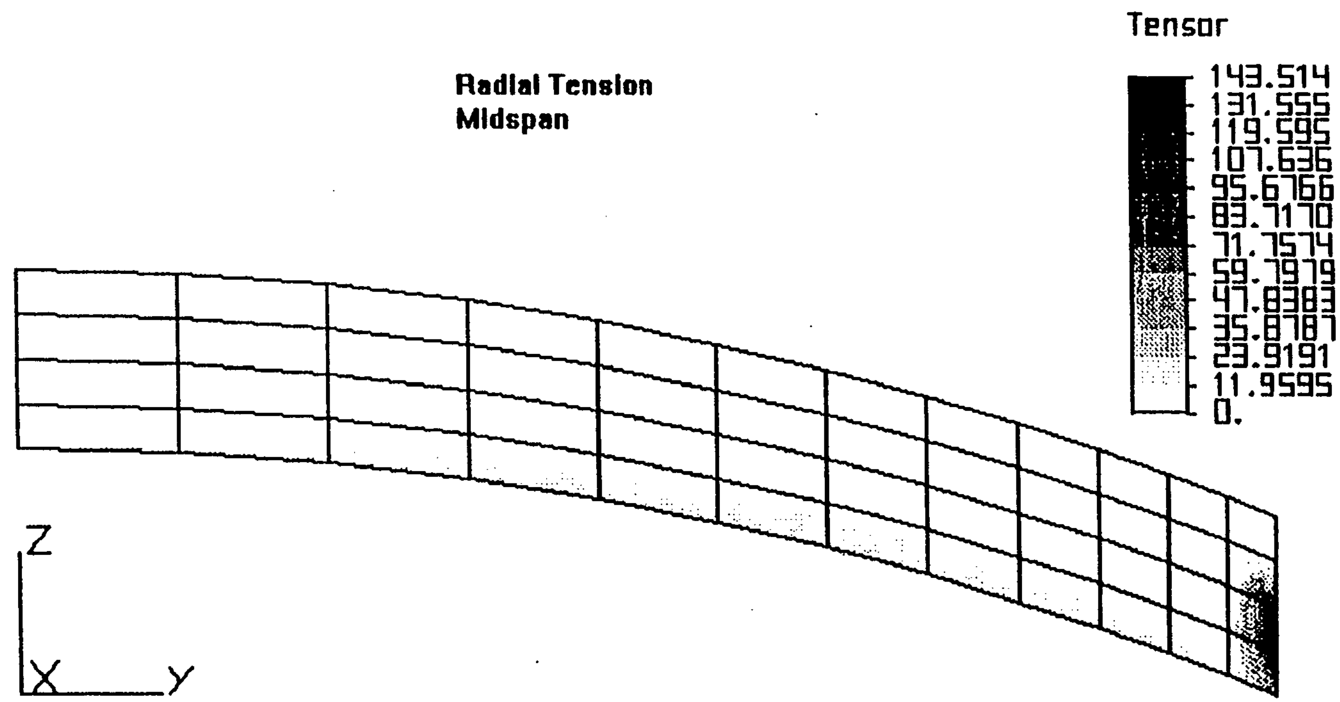
Stresses in the Radial Direction
Brick



Tensile Stresses In the Radial Direction
Brick




Radial Tensile Stress
Brick, Support end
Tensor
143.514
131.555
119.595
107.636
95.6766
83.7170
71.7574
59.7979
47.8383
35.8787
23.9191
11.9595
0.
Z
Y
X


Radial Tension
Midspan
Tensor
143.514
131.555
119.595
107.636
95.6766
83.7170
71.7574
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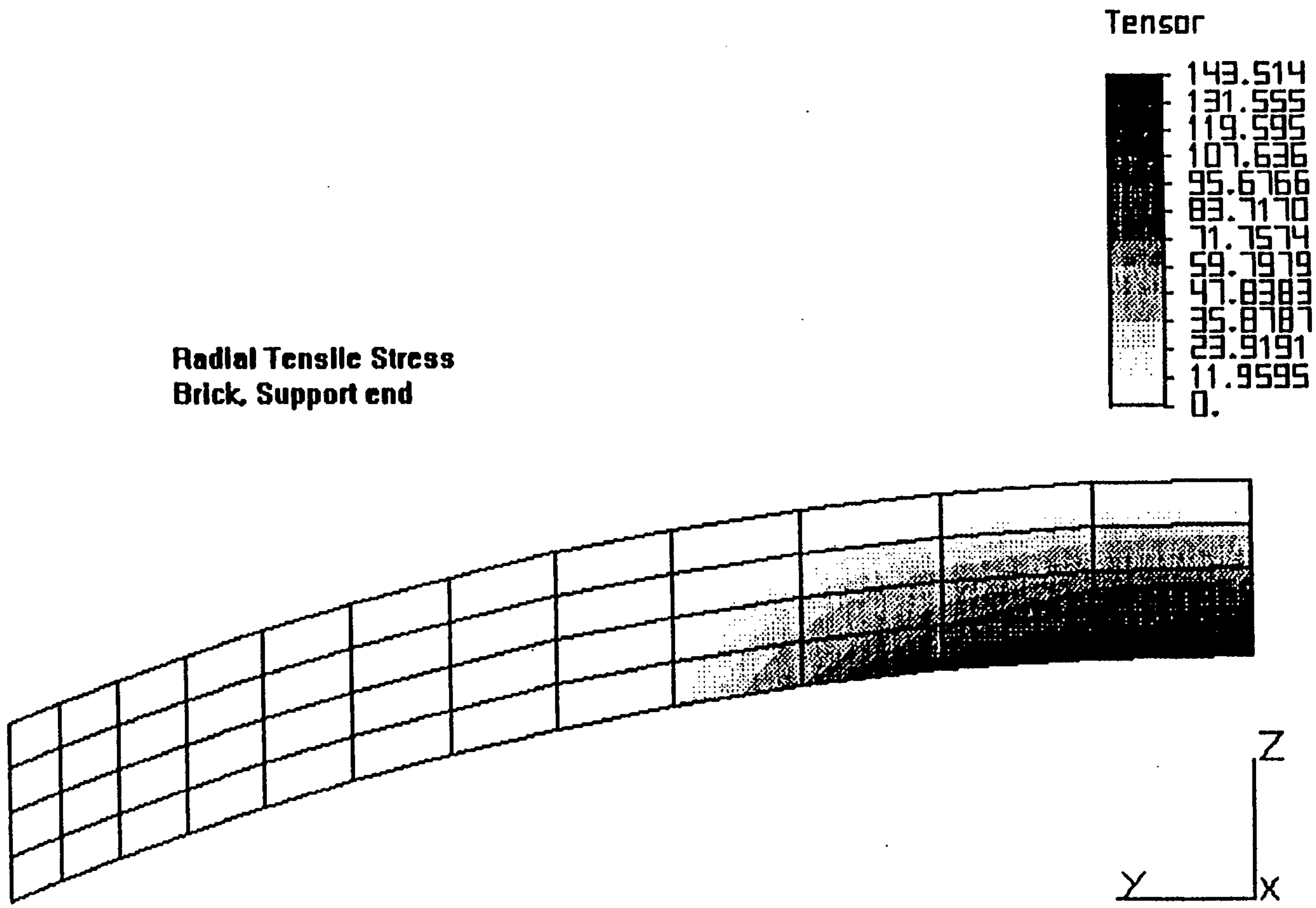
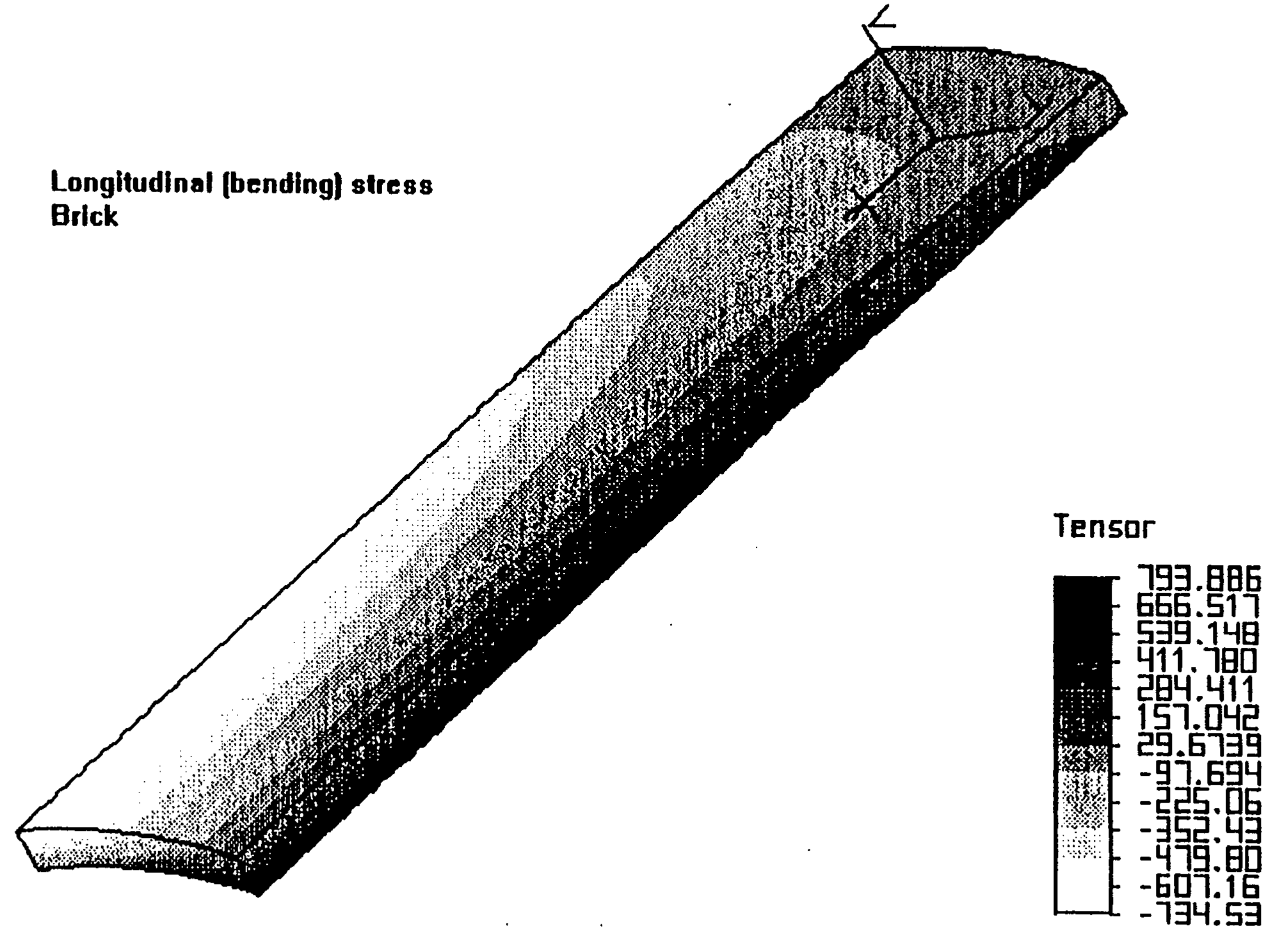
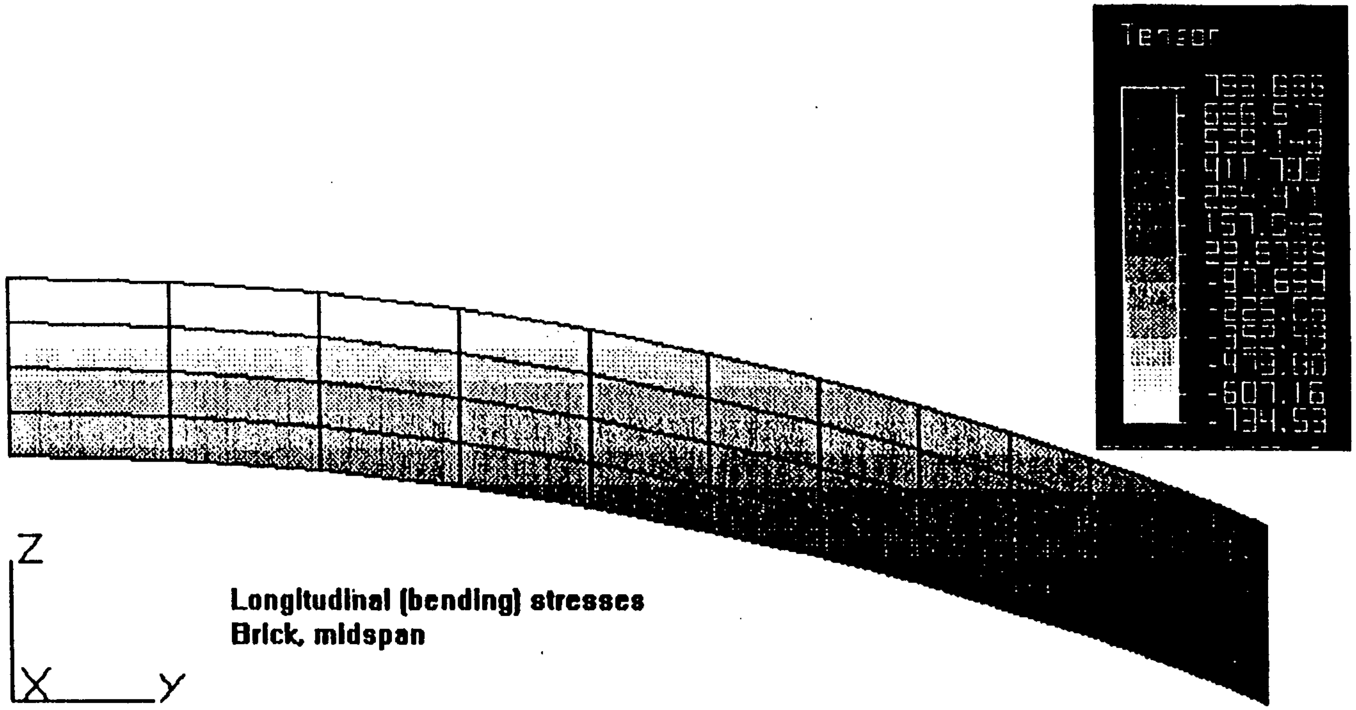
47.8383
35.8787
23.9191
11.9595
0.
Z
X
Y



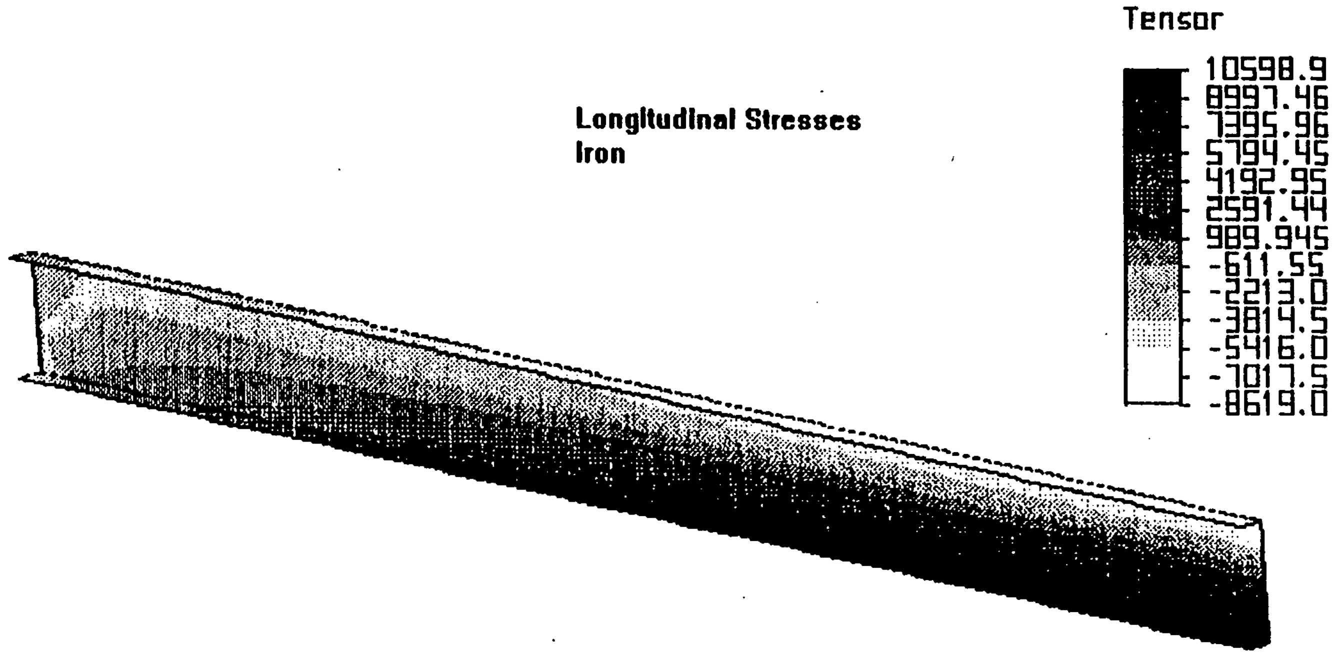
Radial Tensile Stress
Brick, Support end

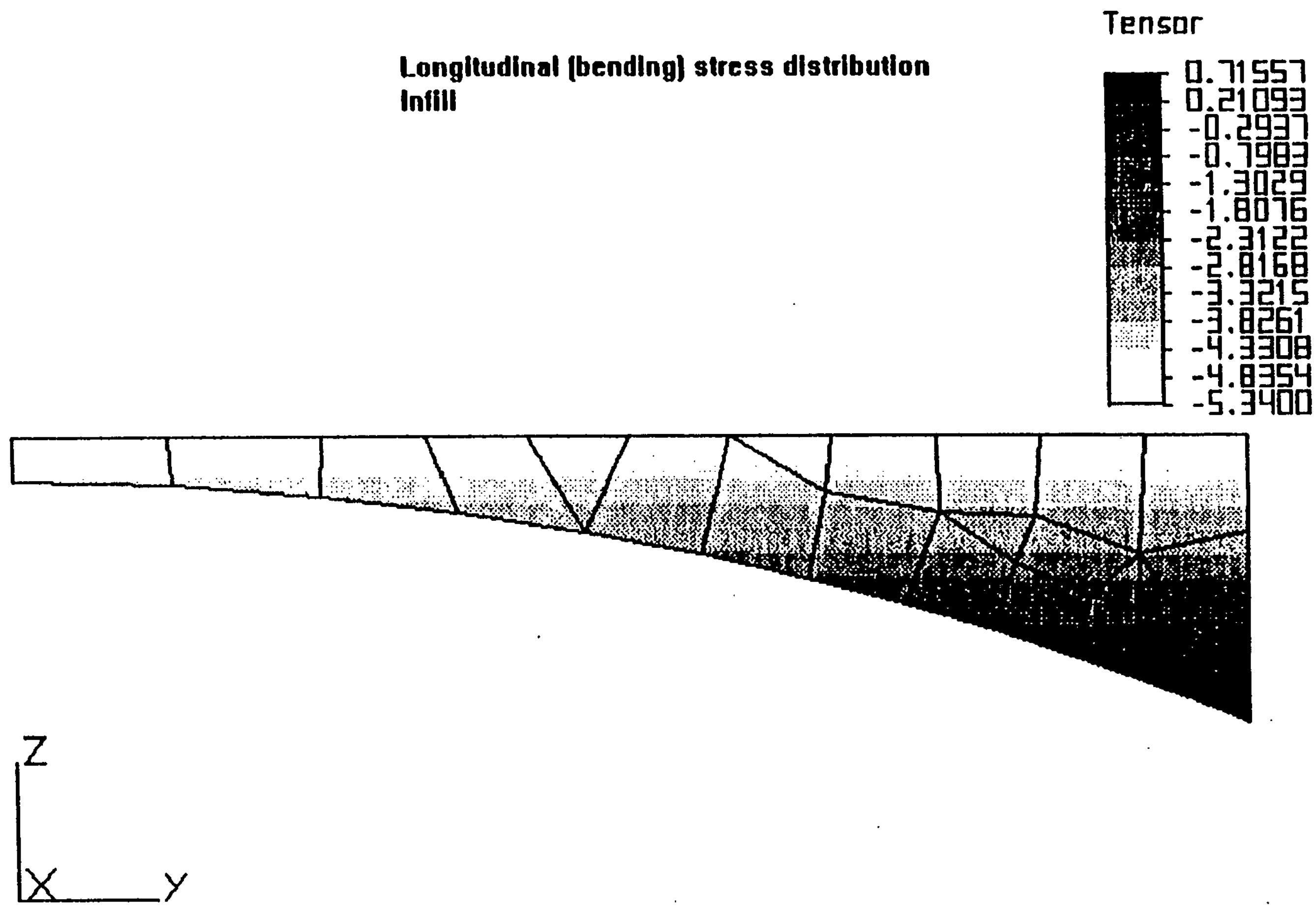


Longitudinal (bending) stress
Brick
Tensor
793.886
666.517
539.148
411.780
284.411
157.042
29.6739
-97.694
-225.06
-352.43
-479.80
-607.16
-734.53



Longitudinal (bending) stresses
Brick, midspan


Longitudinal Stresses
Iron
Tensor
10598.9
8997.46
7395.96
5794.45
4192.95
2591.44
989.945
-611.55
-2213.0
-3814.5
-5416.0
-7017.5
-8619.0


Longitudinal (bending) stress distribution
Infill
Tensor
0.71557
0.21093
-0.2937
-0.7983
-1.3029
-1.8076
-2.3122
-2.8168
-3.3215
-3.8261
-4.3308
-4.8354
-5.3400
Z
X
Y

Cracked Transformed Section Analysis:

To perform this analysis, the cross section was discretized into horizontal slices, 0.1 inch thick. Graph paper was placed over a tracing of the iron's cross section and the width of each slice was measured and loaded into a spreadsheet. To superimpose mathematically the arch on the cross section, sector area formulas for a circle were used. The applied moment was calculated separately as line loads using the appropriate weights for both the construction phase and in-service loads. A linear stress distribution was assumed for both constituents and a crack strength of 340 psi was given to the brick. The forces and moments for each differential area were calculated and summed. To balance the loads, the top strain and neutral axis were guessed until the summations showed good agreement with the applied loads. Shear forces at the supports were also determined.

Conclusions:

Table 8.1 summarizes the results of the analysis and compares them to the allowable stresses set in Chapter 6:

Table 8.1
System Capacity Check

Failure Mode	$\phi\bar{R}$	Stress	
Iron Bending Stress, Yield	26.4ksi	21.8 ksi	ok
Iron Bending Stress, Fracture	38.0ksi	21.8 ksi	ok
Iron, Shear, full cross section	59.4 kip	10,350 lb	ok
Iron, Shear, Cut beams	22.8 kip	10,350 lb	ok
Brick, Crushing due to Thrust*	756 psi	~400 psi	ok
Arch instability*	756 psi	143 psi	ok
Crushing due to bending	792 psi	782 psi	ok
Shear transfer due to masonry bond*	240 psi	103 psi	ok

*results from Finite Element analysis

The stresses in all cases are acceptable and therefore the system can carry a live load of 100 psf. The available uses for this structure are discussed in the next chapter.

Chapter 9: Recommendations

Since it has been established that the structure can withstand a live load of 100 pounds per square foot (psf), the structure can be used for many additional types of gatherings. Prior to this analysis, the corridors were rated for 114 psf due to the capacity of the box beam and the side rooms were rated for 75 psf. According to the *Minimum Design Loads for Buildings and Other Structures*[1] The side rooms (20 ft span) could previously be used for:

Fixed seat assembly areas	60 psf
Bowling alleys, pool rooms and similar recreational areas	75
Reading Rooms	60
Offices	60
Classrooms	40

With the additional rated capacity, these could also be used for:

Assembly and theater areas	
Lobbies	100 psf
Moveable seats	100
Platforms	100
Dance halls and Ballrooms	100
Dining rooms and Restaurants	100
Office Lobbies	100
Retail Stores (first floor)	100

The center spans already carry this kind of capacity, but the additional capacity for the side rooms provides new opportunities for adaptive reuse of the structure. The eminent architect, Tony Wrenn wrote the original Master Plan for West Virginia Independence Hall[2], but many of his

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[1]ANSI/ASCE 7-88, American Society of Civil Engineers, New York, 1990. p 4.

[2]Wrenn, Tony. "Master Plan" Published in Marshall & Marshall's *West Virginia Independence Hall Historic Structure Report.*

recommendations fall under the second category of uses listed above and could not be done. With the additional capacity some modifications to its current use are possible. Wrenn's Master Plan gives many practical and profitable suggestions for usage and are grouped according to area. These suggestions will be reviewed in light of current use and capacity.

Basement. Currently the basement is used as a small theater to show a video about the state of West Virginia. This is because it is one of the few areas that were rated for that use. With other areas now available, this theater could be moved to a more prominent area, providing more gallery space for the basement.

First Floor. Currently the first floor is used as main exhibit space for the museum. Wrenn rightfully recognizes it as "prime public space," and its current use closely follows his suggestions. He also notes, however, that small specialty shops would also be appropriate. This might bring in more tourist traffic as well as provide more visibility in the community.

Second Floor. The original Custom House business room would make an excellent meeting room, as is designated in the Master Plan. With its additional capacity it could be used for seminars, catered luncheons, or meeting areas for civic clubs. In addition, the room is acoustically alive and would be very well suited for chamber ensembles or choral recitals, as well as formal ballroom activities. The office that was used by the first Governor of West Virginia has been restored to its original state as was recommended in the Master Plan and the room next to it has also been renovated into an exhibit concerning the birth of the state. The offices on both ends (other than the governor's office) are well suited to professional office space. One of these offices is currently in use by the State Division of Culture and History. The three other rooms could be used as prime rental space for Architects, Planners, Lawyers or other professionals. This was suggested in the Master Plan, and the use as rental space could help defray some of the cost of rehabilitating these unfinished areas.

Third Floor. The Courtroom has been restored to the desired period and the fixed seating has allowed assembly loads since its restoration. The two offices on the south end are currently used as museum space and additional office space for the DCH. Wrenn notes that these two rooms are the easiest to isolate from the museum traffic and therefore would make prime rental space as well. The unfinished rooms on the North end would also make good office space or museum space, but are more difficult to isolate from the museum traffic. The petit jury room is currently being used for storage. Wrenn suggests that this be the office for the museum, but since the DCH has many archival resources stored at this site in their current office space it is unlikely that this room would be adequate for that purpose, although it is the most isolated area in the structure.

Wrenn is correct to suggest that the commercial use of these rooms carry a fee for gatherings like seminars or sales meetings. This could be an additional source of income for the Foundation that could help the renovations proceed more quickly. Architecturally the structure is an asset to the community and the increased visibility afforded by the additional capacity could make its renovation easier financially.

Further recommendations for future research:

The only reason to increase the capacity of the system would be to use it as a library or stage. If this were necessary, several rehabilitation techniques could be used. For the first floor, decreasing the span length by adding additional columns on the side spans could increase the capacity substantially. If additional capacity were required for long term use in the Custom House business room, the columns in the front of the Post office could be replaced with structural cast iron reproductions that would become loadbearing members, also decreasing the span length.

Wrapping the bottom of the exposed arches in the basement with a fiber reinforced composite could substantially increase the capacity of the floor system supporting the first floor, but without additional inquiry into the capacity of the box beams, the increase would be limited to their 116 psf capacity.

If a future seismic analysis shows the structure to be inadequate, this technique could also assist in adding ductility to the system, thus increasing its seismic capacity.

Chapter 10: Conclusions

The overall conclusion that can be gained from this work is that the structural system has substantially more capacity than was previously assumed and therefore can be used in many more ways than was thought. To arrive at this conclusion it has been established that:

1. From a historical standpoint the structure's design, construction, and use can provide clues to the original designed capacity, any flaws or strengths inherent in the construction, and any cases of overloading. The testing at Trenton confirms that the system has more strength than the I-beams alone and the construction method has a significant impact on the structural analysis of the system. The numerous possible overloading cases also indicate that the structure has significant capacity.

2. Reliability theory can be used to establish reasonable design allowable stresses and loading where building codes are not available. The statistical properties inherent in the materials can be used to assure safe usage designation of the structure.

3. The materials involved were analyzed to determine the statistical properties associated with their material strength and stiffness. Although the results from non-destructive testing is still not conclusive, these techniques did predict the material properties within 20%. As these techniques are refined these results will become suitable for routine field work. It has been suggested that a different frequency sensor might give better results and this will be explored in the future. Destructive testing showed substantial flaws in the material, but also more strength and ductility than was expected.

4. The structural modelling of the system proved more difficult than was assumed. The

system could be re-evaluated when a material non-linear element is formulated for a 20 node solid. The cracked transformed section analysis is sufficient however, showing that the desired capacity is available. In the future, the system should be examined for seismic stability.

Appendix A

Background on Reliability Theory

Design according to statistical reliability dates back to World War II. Prior to that, the minimum elastic limit was determined and reduced by a factor of safety. This allowable stress was then compared to reasonable in-service loads. This method has several advantages. It was relatively easy to use, and existing analysis methods were largely linear. The amount of extra capacity in the system, up to yielding, was well known. With the advent of industrial production, process control provided much more material uniformity and predictability than was previously available with timber, stone, and brick. In addition, materials like iron, steel, and reinforced concrete possessed the ability to continue to carry load even after yielding. This ability to carry load even into the plastic range allows for a completely different set of failure criteria (plastic deformation). Systems could be designed with the ultimate capacity in mind, rather than its elastic (yield) capacity. This allowed for a much more economical design. The uncertainties in material strength and load were accounted for using the concepts of probability. Initial attempts at a practical design methodology used factors to affect either the nominal load or resistance of the system to assure statistical safety. Currently both nominal load and resistance are adjusted (factored) to provide the necessary level of reliability[1].

Although the overall safety factor may be hard to distinguish in a factored design the individual load and resistance factors give a picture of the reliability of the individual steps in design. For instance, from the factors assigned to the load specified by the "Load code", ANSI

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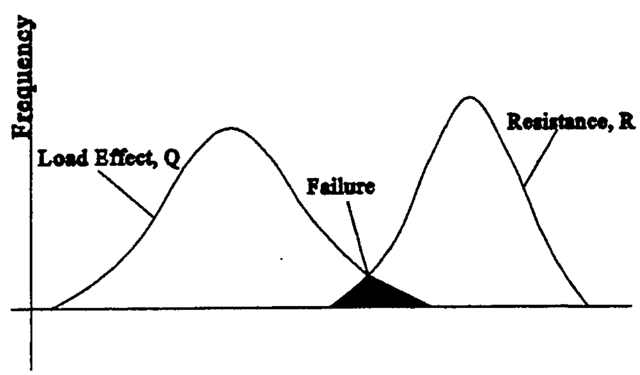
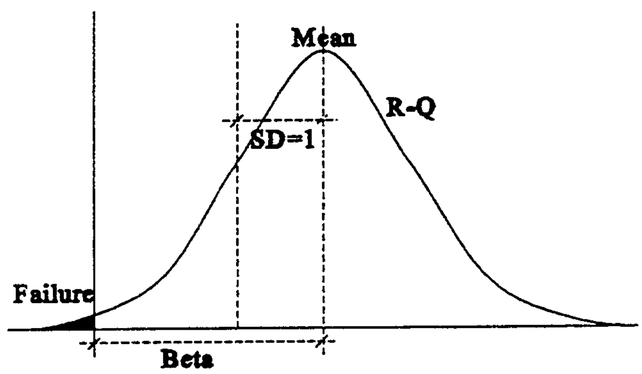
[1]Gambalos, Theodore V. "Reliability of Structural Steel Systems, Structural Engineering Report No. 88-06" University of Minnesota, Minneapolis. August, 1988. pp. 17-18

A58.1/ASCE7-88[2], it is intuitively clear to the designer that the variability in the live load is much higher than that of the dead load because the factor designated by the code is higher for live loads. On the resistance side, different failure modes are examined and assessed penalty factors according to several different failure criteria. Material variability is taken into account, but other effects are also considered, such as the relative importance of a component to the system as a whole, or the ability to recognize a failure before it causes serious consequences. For instance, the resistance factor for a column is much lower than that of a beam in bending because the consequences of failure are of a different magnitude, and there may be less warning of failure.

Both load and resistance factors derive from an overall reliability index, β. It is relatively simple to construct probability histograms for both the capacity of a component, (R) and its loading, (Q). When graphed on the same axis, the overlap between the two curves indicates failure. Increasing resistance capacity shifts the second curve to the right, reducing the overlap and therefore reducing the probability of failure. Another way to describe the probability of failure is to produce a **reliability histograph**. This is constructed by





Reliability Histograph

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[2]*Minimum Design Loads for Buildings and Other Strucutres*. ANSI/ASCE 7-88. American Society of Civil Engineers, New York, 1990.

subtracting the loading histogram from the capacity histogram (R-Q). The curve is then normalized by dividing by the standard deviation. Areas of the curve that are greater than zero indicate a safe situation--capacity exceeds load. The area of the curve that extends into the negative region indicates failure of that limit state. The reliability index, β, is a measure of how many standard deviations (σ^2), on the reliability histogram, that the mean exceeds zero. As β increases, the probability of failure decreases. Typical β values for bending members in steel range around 2.5. On the other hand, the β value used for steel connections is 3.5, indicating the intuitive judgment that the reliability of the connection is more crucial than that of the member[3]. In general, the value of the reliability index should be linked to the consequences of failure.

A Level I reliability analysis establishes individual load and resistance factors to assure that the design will meet or exceed minimum reliability. It is based on the Equation:

$$\phi R_n \geq \Sigma \gamma_i Q_{ni}$$

Where: ϕ is the resistance factor

R_n is the nominal resistance

γ is the load factor

Q_n is the nominal load effect

The typical practitioner will use these factors as given by the governing codes and specifications and is assured that his design is therefore "safe." A certain measure of legal

[3]Gambalos, Theodore V. "Reliability of Structural Steel Systems, Structural Engineering Report No. 88-06" University of Minnesota, Minneapolis, August, 1988

protection is provided by the agreement of safe practice embodied in the codes and subsequently adopted by the local building authorities. This type of design methodology is labeled a Level I reliability analysis, and is adequate for a practicing engineer to use in typical design situations. Level I factors are not, however, chosen without analytical and practical rationale. These factors are typically chosen through a Level II reliability analysis that assumes either a normal or lognormal probability distribution for most variables and uses the mean and standard deviation to determine the reliability index of the structure. In an analysis, this is then compared to a minimum reliability index desired. Level III analysis replaces the assumption of normal or lognormal distributions with assumed distributions that more closely approximate the actual distribution functions. This kind of analysis is often reserved for mechanical engineering designs that have very tight operational requirements, or new and obsolete historic materials in unusual situations. Although the limit state codes are largely derived from Level II and Level III statistical analysis of typical building systems, reliability indices are not arbitrarily selected. Previous safe practice has always been the foundation for engineering judgment, therefore target reliability indices are initially selected to correlate with the measured reliability of successful design practice. As systems are shown to be successful, reliability indices gradually reduce. Code writing agencies typically will only increase reliability indices because of a pattern of system failures, although one catastrophic failure can cause the same industry reaction.

Appendix B

Reliability Index Selection

Although, from a design standpoint, the overall reliability of the system is dependant on both the load and resistance factors applied, current practice typically involves adjusting only the resistance factors. The target reliability index used for non-environmental loads in ASCE7-88 is 3.0, which is an average reliability for most systems and components[1]. Adjusting only resistance factors toward a target β will slant the overall reliability toward 3.0, but this is acceptable given the uncertainties involved. Allowing the load factors to remain uniform, with only minor code adjustments, provides for simplicity of practice in Level I design and analysis and may reduce error on the part of the designer.

There has been little effort made in attempting to quantify the reliability of a historic structure[2], either as a system or for its individual components. Although the load assumptions and factors used in Level I (Designer level) Limit State Design are roughly applicable, the resistance assumptions are not valid as they are based on the statistical properties of modern materials like steel and concrete. Level II Limit State Design Methods, like First Order Second Moment (FOSM) analysis, can be effectively used in their place, but they require information on the mean and standard deviation of the limit state and are beyond the abilities of most engineering practitioners. In addition, the minimum reliability index should be chosen with care in light of several factors: the consequences of failure, redundancy, importance, ductility, and the amount of

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[1]Gambalos, Theodore V. "Reliability of Structural Steel Systems, Structural Engineering Report No. 88-06" University of Minnesota, Minneapolis, August, 1988

[2]Yao, JTP Safety and Reliability of Existing Strutures Pitman Press. Boston, 1985.

available information concerning the material properties. Each of these factors influincing reliability will now be discussed in detail.

Consequences of Failure. The consequences of failure are directly linked to the parallel or series nature of the system. In designing the LRFD specification the consequences of failure for that limit state, in part, dictated the target β for that component limit state. Table B.1 below correlates the consequences of failure with the appropriate reliability index[3]:

Damage	β	Approx. ϕ	Consequences of Failure
None	1.5		Serviceability
Slight	2.0	1.00	Component failure, no system failure
Moderate	2.5	.90	Component failure, less than substantial system failure
Serious	3.0	.80	Some system failure, a typical design failure
Complete	3.5	.70	Complete destruction, no loss of life
Complete	4.0	.65	Complete destruction, loss of life

Table B.1 Reliability Index and Consequences of Failure

Redundancy. The level of redundancy in a structure can have a positive or negative effect on its overall reliability. In a parallel system, the system failure only occurs when all or most of the components fail, and redundancy in the system increases its reliability substantially. On the other hand, in a series system the failure of one component can result in the failure of other components or of the system as a whole, and additional components will only reduce the overall reliability. Therefore a series system requires an increase in the minimum reliability index. It is important to note that series systems rarely have a 1 to 1 correlation between failure of components. A small drop in the correlation of failure between components in a highly interdependent system can

[3]Gambalos, Theodore V. "Reliability of Structural Steel Systems, Structural Engineering Report No. 88-06" University of Minnesota, Minneapolis, August, 1988

significantly increase the reliability of the system, and allow for a lower minimum reliability index.

Importance. ASCE 7-88/ANSI A58.1 defines the Importance Factor, I, as "a factor that accounts for the degree of hazard to human life and damage to property" that is applied to the environmental design loads. Table B.2 lists the 4 catagories of importance[4] and their corresponding multiplication factors for wind, snow and seismic loads:

Cat.		Wind[5]	Snow	Seismic
I	All buildings and structures except listed below	1.00	1.0	1.0
II	Buildings and structures where the primary occupancy is one in which more than 300 people congregate in one area.	1.07	1.1	1.25
III	Buildings and structures designated as essential facilities [in the case of an emergency]	1.07	1.2	1.5
IV	Buildings and structures that represent a low hazard to human life in the event of failure, such as agricultural buildings, certain temporary facilities, and minor storage facilities.	0.95	0.8	N/A

Table B.2 Importance Factors

An increase in importance factor substantially increases the reliability index of the system. In ASCE7-88, importance factors are used to increase the inherent reliability index of a design.

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[4]ANSI/ASCE 7-88, *Minimum Design Loads for Buildings and Other Structures.* American Society of Civil Engineers, New York, 1990. p 2.

[5]The factors listed are for structures 100 miles or more from hurricane oceanlines. Factors are higher at the oceanline, and are interpolated inbetween. From ASCE7-88/ANSI A58.1

For example, the reliability index for snow loading increases from 3.0 to 3.5 and then to 3.9 for categories I, II and III respectively.[6]

Structural importance factors are typically applied to prevent loss of life and essential property. No Importance factor is applied based on the historic importance of a structure. Since the load side of the equation is directly influenced by regional and sociological factors like geographic location, use, and occupancy type, it is logical that sociological importance be addressed as a load adjustment. These load adjustments, however, cannot be taken lightly. As mentioned earlier, an increase of 10% in the required capacity can increase the reliability index by half a point, reducing the chance of failure by nearly an order of magnitude. The structural increase required to attain that reliability level may not be justifiable except for the most important landmarks because of the additional cost associated with the increased reliability. Structures designated as National Historic Landmarks should probably be assessed a Category II importance factor due to the potential impact of their loss on society. Although listing on the National Register of Historic Places recognizes the importance of a structure in the context of national and local history, listed properties are probably not of sufficient sociological stature to justify the economic impact associated with maintaining that level of reliability. Many National Register listed properties are under economic hardship already. Increasing structural requirements on those properties may require unnecessary strengthening that could make a renovation or adaptive reuse of the site unattractive.

Ductility. Engineering design attempts to provide for ductile failures in most systems. This allows for at least "running time" to escape collapse and, possibly, correction time to prevent

[6]ibid.

serious damage. Although collapse is extremely rare, the consequences are overly severe and should be avoided. Less serious failures linked to serviceability concerns are far more frequent. These should also be avoided, since they can render the structure unusable. However, these failures are not usually included in establishing target β values. They are frequently considered later in the design. Although in current practice it is rare to see a non-ductile system, in the analysis of historic structures it is not at all uncommon. Cast iron, stone and brick are all extremely brittle materials. In addition, wrought iron, which is usually assumed as ductile, may not be due to poor manufacturing processes, overwork or impurities in the alloy. In such systems, the β values must be increased because of the lack of warning of catastrophic failure.

Amount of available information on statistical material properties. With historic materials, the engineer must consider the fact that many of these materials were ground-breaking at the time, and the manufacturers may not have had the experience to turn out a reliable, high quality product. Even if the material was of good quality initially, many materials, like concrete and brick, degrade with time. It is crucial, therefore, to be aware of the available destructive and nondestructive tests on the material considered. These can be used to establish resistance histograms to be used with typical load histograms.

Where there is statistical information from numerous samples, or sample areas, the minimum reliability index can be reduced because of the amount of information about the material behavior. Where assumptions must be made because of lack of verified material properties, the reliability index must remain high. Even statistical properties derived from a few tests can substantially increase the confidence in the design and allow for a less stringent reliability index.

As a comparision, typical β values for several failure modes are listed below along with an approximate rating for some of the criteia previously discussed:

Failure Mode[7]	Damage	Series/ Parallel	Ductility/ Warning	β
Reinf. Concrete (R/C) Beam, Gr 40 or 60, med ρ	Serious	P	Good	2.8
Short tied R/C column, compression failure	Complete	S	Fair	3.4
Short spiral R/C column, compression failure	Serious	S	Good	3.0
Shear strength of R/C beam (2x min. stirrups)	Moderate	S/P	Fair	2.4
Steel tension member, yield	Moderate	S/P	Good	2.5
Steel tension member, fracture	Complete	S/P	Good	3.4
Compact steel beam	Serious	P	Excellent	3.1
Steel Column, λ=0.5	Complete	S	Excellent	3.1
Fillet Welds	Complete	S	Poor	3.9
A325 Bolts, shear	Complete	S/P	Fair	4.4
Aluminum columns (L/D=5)	Serious	S	Excellent	2.8
Brick Walls, unreinforced, inspected	Serious	P	Poor	7.0
Brick Walls, unreinforced, uninspected	Serious	P	Poor	4.2

Table B.3--Typical current Failure modes and β values.

[7]AISC. *Load and Resistance Factor Design, Ed.1.* AISC, 1986. p.6-146.